

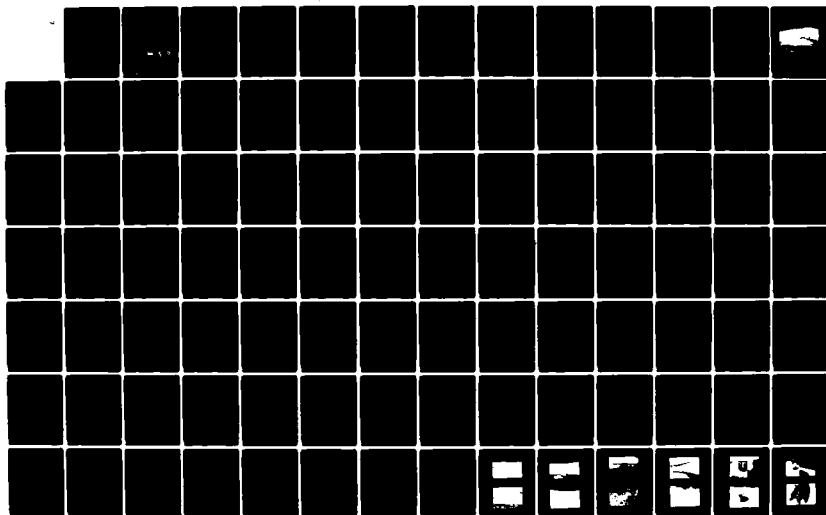
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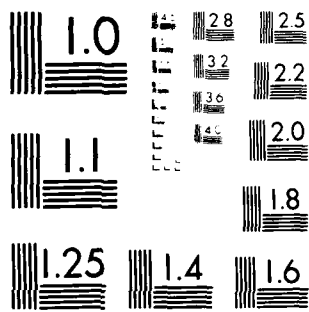
NATIONAL DAM INSPECTION PROGRAM MIANUS RESERVOIR DAM
(CT 00050) CONNECTIC. (U) CORPS OF ENGINEERS WALTHAM MA
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CONNECTICUT WESTERN COASTAL AREA
STAMFORD, CONNECTICUT

**MIANUS RESERVOIR DAM
CT 00050**

**PHASE I INSPECTION REPORT
NATIONAL DAM INSPECTION PROGRAM**



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DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION, CORPS OF ENGINEERS
WALTHAM, MASS. 02154

SEPTEMBER 1978

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20. ABSTRACT (Continue on reverse side if necessary and identify by block number) The Mianus Reservoir Dam consists of an earth embankment with a selected impervious fill core, drop inlet spillway, stilling basin, a wasteway having a fuse plug and pumping facilities. The dam section is 950 ft. long with a maximum height of 75 ft. The top of the dam is 20 ft. wide The downstream side slopes of the earth embankment are 2 horizontal to 1 vertical. The upstream slope is 2-1/2 horizontal to 1 vertical. Riprap is in place on the upstream face. The morning glory-type spillway is 8 ft. in diameter.		



DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION, CORPS OF ENGINEERS
424 TRAPELO ROAD
WALTHAM, MASSACHUSETTS 02154

REPLY TO
ATTENTION OF:
NEDED

JUN 11 1979

Honorable Ella T. Grasso
Governor of the State of Connecticut
State Capitol
Hartford, Connecticut 06115

Dear Governor Grasso:

I am forwarding to you a copy of the Mianus Reservoir Dam Phase I Inspection Report, which was prepared under the National Program for Inspection of Non-Federal Dams. This report is presented for your use and is based upon a visual inspection, a review of the past performance and a brief hydrological study of the dam. A brief assessment is included at the beginning of the report. I have approved the report and support the findings and recommendations described in Section 7 and ask that you keep me informed of the actions taken to implement them. This follow-up action is a vitally important part of this program.

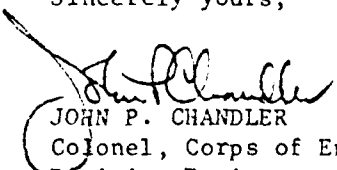
A copy of this report has been forwarded to the Department of Environmental Protection, the cooperating agency for the State of Connecticut. In addition, a copy of the report has also been furnished the owner, Connecticut-American Waterworks Company, Inc., 125 East Putnam Avenue, Greenwich, Connecticut 06830, ATTN: Mr. Joseph O. Yates, Jr., Manager.

Copies of this report will be made available to the public, upon request, by this office under the Freedom of Information Act. In the case of this report the release date will be thirty days from the date of this letter.

I wish to take this opportunity to thank you and the Department of Environmental Protection for your cooperation in carrying out this program.

Sincerely yours,

Incl
As stated


JOHN P. CHANDLER
Colonel, Corps of Engineers
Division Engineer

MIANUS RESERVOIR DAM

CT 00050

CONNECTICUT WESTERN COASTAL AREA

STAMFORD, CONNECTICUT

PHASE I INSPECTION REPORT

NATIONAL DAM INSPECTION PROGRAM



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PHASE I REPORT
NATIONAL DAM SAFETY PROGRAM

Name of Dam: MIANUS RESERVOIR DAM

State Located: Connecticut

County Located: Fairfield County

Stream: Mianus River

Date of Inspection 3 AUGUST 1978

BRIEF ASSESSMENT

The Mianus Reservoir Dam consists of an earth embankment with a selected impervious fill core, drop inlet spillway, stilling basin, a wasteway having a fuse plug and pumping facilities. The dam section is 950 feet long with a maximum height of 75 feet. The top of the dam is 20 feet wide. The downstream side slopes of the earth embankment are 2 horizontal to 1 vertical. The upstream slope is 2-1/2 horizontal to 1 vertical. Riprap is in place on the upstream face. The morning glory-type spillway is 8 feet in diameter.

Based on the visual inspection, records available, and past operational performance, the dam is judged to be in good condition.

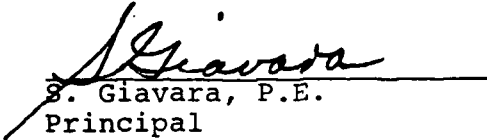
The project will not pass the test flood without overtopping the dam, and therefore, the total spillway capacity is inadequate. The spillway capacity is judged seriously inadequate since the project will not pass one-half the test flood without overtopping the dam (35 percent). The test flood will overtop the dam by approximately 2.9 feet.

It is recommended that the owner ensure that the materials contained in the fuse plug are excavated and replaced, without tamping. Additionally, the scour hole at the spillway

should be repaired. The owner should control trespassing on the dam, particularly near the junction of the left abutment and the downstream slope where trespassing is extensive at the present time.

Round-the-clock surveillance should be provided during periods of high precipitation. The owner should develop a formal warning system. An operational procedure to follow in the event of an emergency should also be adopted.

Recommendations and remedial measures described should be implemented by the owner within 2 years after receipt of this Phase I Inspection Report.


S. Giavara, P.E.
Principal

Registered, CT 7634

This Phase I Inspection Report on Mianus Reservoir Dam has been reviewed by the undersigned Review Board members. In our opinion, the reported findings, conclusions, and recommendations are consistent with the Recommended Guidelines for Safety Inspection of Dams, and with good engineering judgment and practice, and is hereby submitted for approval.



CHARLES G. TIERSCH, Chairman
Chief, Foundation and Materials Branch
Engineering Division

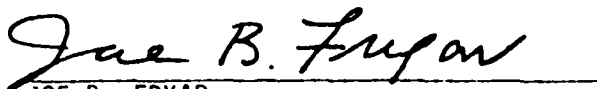


FRED J. RAVENS, Jr., Member
Chief, Design Branch
Engineering Division



SAUL COOPER, Member
Chief, Water Control Branch
Engineering Division

APPROVAL RECOMMENDED:



JOE B. FRYAR
Chief, Engineering Division

PREFACE

This report is prepared under guidance contained in the Recommended Guidelines for Safety Inspection of Dams, for Phase I Investigations. Copies of these guidelines may be obtained from the Office of Chief of Engineers, Washington, D.C. 20314. The purpose of a Phase I Investigation is to identify expeditiously those dams which may pose hazards to human life or property. The assessment of the general condition of the dam is based upon available data and visual inspections. Detailed investigation, and analyses involving topographic mapping, subsurface investigations, testing, and detailed computational evaluations are beyond the scope of a Phase I investigation; however, the investigation is intended to identify any need for such studies.

In reviewing this report, it should be realized that the reported condition of the dam is based on observations of field conditions at the time of inspection along with data available to the inspection team. In cases where the reservoir was lowered or drained prior to inspection, such action, while improving the stability and safety of the dam, removes the normal load on the structure and may obscure certain conditions which might otherwise be detectable if inspected under the normal operating environment of the structure.

It is important to note that the condition of a dam depends on numerous and constantly changing internal and external conditions, and is evolutionary in nature. It would be incorrect to assume that the present condition of the dam will continue to represent the condition of the dam at some point in the future. Only through continued care and inspection can there be any chance that unsafe conditions be detected.

Phase I inspections are not intended to provide detailed hydrologic and hydraulic analyses. In accordance with the established Guidelines, the Spillway Test flood is based on the estimated "Probable Maximum Flood" for the region (greatest reasonably possible storm runoff), or fractions thereof. Because of the magnitude and rarity of such a storm event, a finding that a spillway will not pass the test flood should not be interpreted as necessarily posing a highly inadequate condition. The test flood provides a measure of relative spillway capacity and serves as an aid in determining the need for more detailed hydrologic and hydraulic studies, considering the size of the dam, its general condition and the downstream damage potential.

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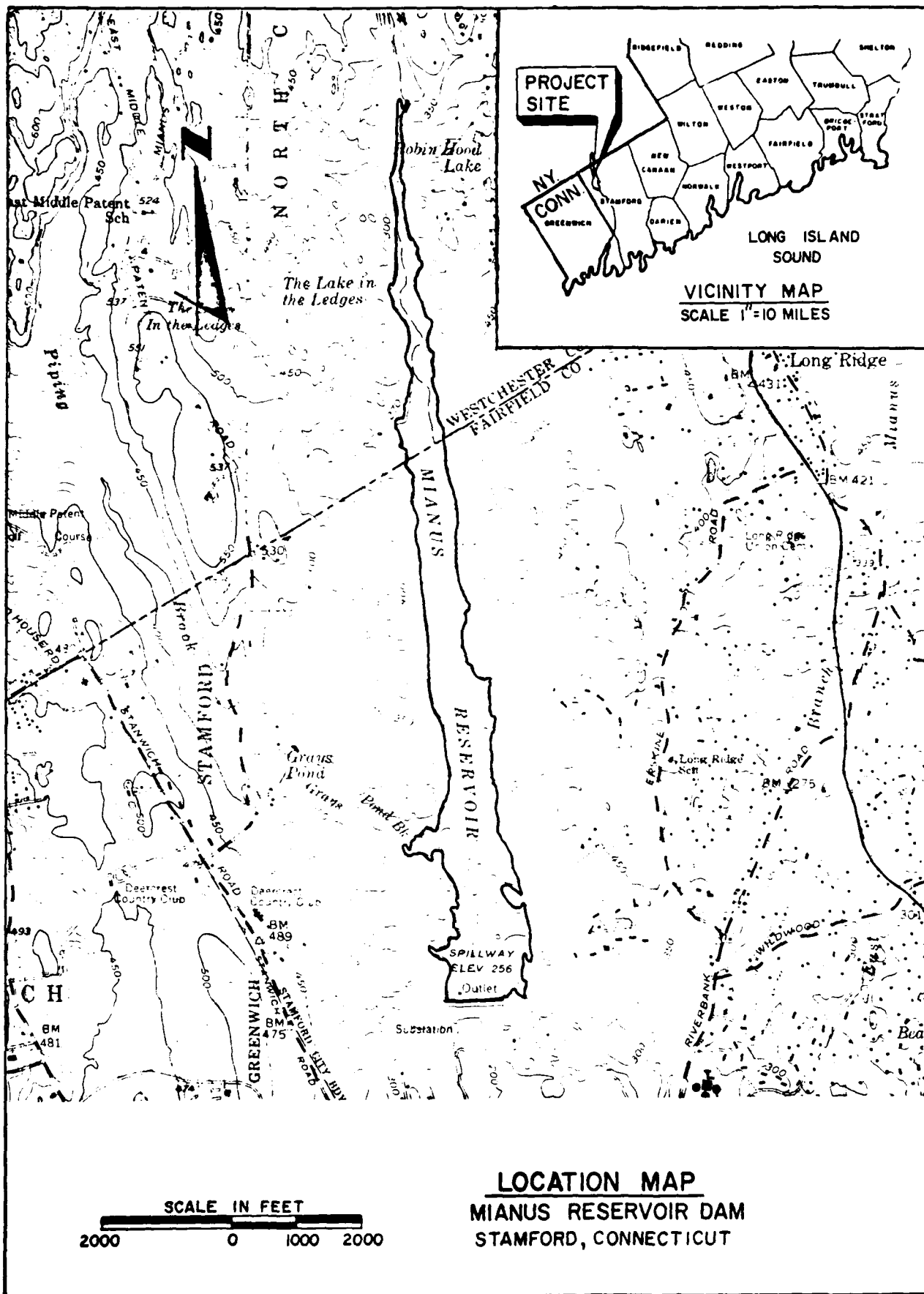
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<u>Appendix</u>	<u>Description</u>
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B	Engineering Data
C	Photographs
D	Hydrologic Computations
E	Information - National Inventory of Dams



MIANUS RESERVOIR DAM



PHASE I INSPECTION REPORT
MIANUS RESERVOIR DAM CT 00050

SECTION 1 - PROJECT INFORMATION

1.1 GENERAL:

a. Authority. Public Law 92-367, August 8, 1972, authorized the Secretary of the Army, through the Corps of Engineers, to initiate a national program of dam inspection through the United States. The New England Division of the Corps of Engineers has been assigned the responsibility of supervising the inspection of dams within the New England Region. Flaherty Giavara Associates, P.C. has been retained by the New England Division to inspect and report on selected dams in the State of Connecticut. Authorization and notice to proceed was issued to Flaherty Giavara Associates, P.C. under a letter of 25 April 1978 from Ralph T. Garver, Colonel, Corps of Engineers. Contract No. DACW33-78-C-0309 has been assigned by the Corps of Engineers for this work.

b. Purpose.

1) Perform technical inspection and evaluation of non-federal dams to identify conditions which threaten the public safety and thus permit correction in a timely manner by non-federal interests.

2) Encourage and assist the States to initiate quickly effective dam safety programs for non-federal dams.

3) To update, verify and complete the National Inventory of Dams.

1.2 DESCRIPTION OF PROJECT:

a. Description of Dam and Appurtenances. This dam popularly known as the Samuel J. Barch Reservoir Dam consists of an earth embankment with a selected impervious fill core, drop inlet spillway, stilling basin, a wasteway having a fuse plug and pumping facilities, built in 1955. The dam section is 950 feet long with a maximum height of 75 feet. The top of the dam is 20 feet wide. The downstream side slopes of the earth embankment are 2 horizontal to 1 vertical. The upstream slope is 2-1/2 horizontal to 1 vertical. Riprap is in place on the upstream face. The morning glory-type spillway is 8 feet in diameter.

b. Location. The dam is located approximately 2-1/2 miles north of the Merritt Parkway in the Town of Stamford on Mianus River within the Connecticut western coastal area.

c. Size Classification. The applicable guideline indicates that for an intermediate category the storage in acre-feet for the impoundment must be greater than or equal to 1,000 and less than 50,000. The size classification may be determined by either storage or height, whichever gives the larger size category. Based on the storage capacity of the dam, the size classification is intermediate. The top of dam storage for Mianus Reservoir Dam is 7,515 acre-feet.

d. Hazard Classification. The dam is classified as having a high hazard potential. This classification is based on the 10 or more houses situated along the narrow valley which would be affected by a dam failure flood wave.

e. Ownership. Mianus Reservoir Dam is owned by the Connecticut-American Waterworks Company, Inc. - Greenwich District.

f. Purpose of Dam. The dam was constructed to form an impounding reservoir. The reservoir forms part of the water company's supply and distribution system, providing potable water to the residents of Greenwich.

g. Design and Construction History. The dam was completed in 1955. The American Water Works Service Company, Inc. designed the dam and its appurtenances under the direction of Howard J. Carlock. The dam was constructed by Merritt-Chapman and Scott Corporation, New York, New York. Any subsequent modifications are unknown.

h. Normal Operating Procedures. A 20" lock joint pipe transports water from Brush Dam to the stilling basin at Mianus Reservoir from which it flows downstream to the Mianus Filter Plant. Just upstream of the stilling basin, pumping facilities withdraw a portion of the water from the 20" lock joint pipe and pumps water to Rockwood Lake.

1.3 PERTINENT DATA:

- a. Drainage Area - 18.3 sq. miles
- b. Discharge at Dam Site -
- | | |
|--------------------------------------|-----------------------------|
| Maximum Known Flood | Unknown |
| Warm Water Outlet | Not Available |
| Div. Tunnel Low Pool Outlet | Not Available |
| Diversion Tunnel Outlet | Not Available |
| Gated Spillway | None |
| Ungated Spillway at Max. Pool | 2,200 CFS @ 1 Ft. freeboard |
| Total Spillway Cap. at Max. Pool | 3,000 CFS @ no freeboard |
| Emergency Wasteway without Fuse Plug | 2,900 CFS @ 1 Ft. freeboard |
| Emergency Wasteway without Fuse Plug | 3,900 CFS @ no freeboard |
- c. Elevation (above M.S.L.) -
- | | |
|---------------------------------------|---------------|
| Top of Dam | 262 |
| Max. Design Pool | Not Available |
| Full Flood Control Pool | Not Available |
| Recreation Pool | Not Available |
| Spillway Crest Ungated | 252 |
| Upstream Portal Invert. Div. Tunnel | 188 |
| Downstream Portal Invert. Div. Tunnel | 188 |
| Streambed at Centerline of Dam | 183 |
| Maximum Tailwater | Unknown |
- d. Reservoir -
- | | |
|------------------------------|----------------|
| Length of Max. Pool | 13,500 feet |
| Length of Recreation Pool | Not Applicable |
| Length of Flood Control Pool | Not Applicable |
- e. Storage -
- | | |
|--------------------|-----------------|
| Recreation Pool | Not Applicable |
| Flood Control Pool | Not Applicable |
| Design Surcharge | Not Applicable |
| Top of Dam | 7,515 Acre-Feet |
- f. Reservoir Surface (acres) -
- | | |
|--------------------|----------------|
| Top of Dam | Not Available |
| Max. Pool | Not Available |
| Flood Control Pool | Not Applicable |
| Recreation Pool | Not Applicable |
| Spillway Crest | 230 |
- g. Dam -
- | | |
|------------|---------------------------------|
| Type: | Earth embankment, concrete core |
| Length: | 950 feet |
| Height: | 75 feet |
| Top width: | 20 feet |

Side Slopes: Downstream: 1 vertical to 2 horizontal
Upstream: 1 vertical to 2-1/2 horizontal
Zoning: Selected impervious fill core
Impervious core: Selected fill
Grout Curtain: Unknown

h. Diversion and Regulating Tunnel -

Type: Cast iron pipe
Length: 500 feet
Diameter: 30 inches
Access: From stilling basin
Regulation: Butterfly valve

i. Spillway -

Type: 8' diameter conc. morning glory
Length of Weir: 25 feet
Crest Elevation: 252
Gates: Ungated
Upstream Channel: Reservoir
Downstream Channel: Natural Streambed
Spillway is founded on: Rock

j. Regulating Outlets -

Gates: None
Conduits: 30" diameter cast iron pipe to stilling basin

SECTION 2 - ENGINEERING DATA

2.1 DESIGN:

The design of the Dam was made by American Waterworks Service Company, Inc., Philadelphia, Pennsylvania. The plans were prepared by Howard C. Carlock, P.E., Connecticut Registration Number 3286.

The principal engineering data available are:

a. Project 18 - "Contract Documents for the Construction of a Dam and Impounding Reservoir on the Mianus River for the Greenwich Water Company, Greenwich, Connecticut," January, 1956.

b. Project 18 - Contract Drawings Nos. 36-268-1 through 36-268-8 (see Appendix VI).

c. Greer and McClelland, "Investigation and Design of the Mianus River Dam, Stamford, Connecticut," November, 1954.

d. Haley and Aldrich, "Report on Stability and Seepage Analysis, Mianus River Dam, Stamford, Connecticut," April, 1959 (see Appendix V).

e. Woodward-Clyde-Sherard and Associates, "Field and Laboratory Investigation, S. J. Bargh Reservoir (Mianus River) Dam, Stamford, Connecticut," January 19, 1961.

f. Haley & Aldrich, "Letter Report - S. J. Bargh Reservoir (Mianus River) Dam," July 6, 1961.

g. Observation well readings for five wells installed along the downstream slope for the following dates: July 8, 1965, July 1, 1966, November 18, 1968, and August 3, 1978.

2.2 CONSTRUCTION:

No information is available concerning the foundation preparation or embankment construction.

2.3 OPERATION:

No formal operation records are available.

2.4 EVALUATION:

a. Availability. The information available concerning the embankment consists of a design cross section and the identification of the embankment materials as "impervious core wall" and "random fill." Limited engineering data is available concerning the properties of the random fill to a depth of 5 feet. Six test pits were excavated on October 26, 1960, and the field results are reported in Reference e, Subsection 2.1. No additional engineering data are available concerning the properties of other materials in the dam. No information is available about the foundation materials encountered during the construction of the embankment sections.

b. Adequacy. The available data are not sufficient to evaluate the soils in the core and shells and in the foundation of the dam. The evaluation must be based primarily on the results of the visual inspection. The information available is considered adequate for the purposes of a Phase I investigation.

No conflicts have been noted between the available data and the observations made during the inspection. In general, there is no reason to question the validity of the available data.

SECTION 3 - VISUAL INSPECTION

3.1 FINDINGS:

a. General. The dam appears to be in good condition and well maintained. No seepage was observed. Vertical and horizontal alignment of the dam was good. No lateral movement or settlement of the crest was noted. Vehicular paths and footpaths exist at several locations on the downstream slope. Some local deterioration of the riprap has occurred.

b. Dam. The upstream slope is covered with riprap from below the water surface to the crest of the dam. The riprap was in good condition except for some surface erosion and deterioration and small windows exposed through the riprap.

The crushed stone which covers the majority of the crest is generally in good condition and well maintained.

Several bike and footpaths were located on the downstream slope from the toe to the crest of the dam. Vehicular traffic has worn a path at the junction between the left abutment and the downstream face of the dam. The downstream slope was covered with grass and was generally well maintained. No animal holes were located on the downstream slope at the time of inspection.

The groundwater elevations obtained on August 3, 1978 in the five observation wells in the downstream embankment, located as shown on Figure 4, Appendix B, are indicated below:

GROUNDWATER OBSERVATIONS

<u>PIPE NO.</u>	<u>DEPTH TO WATER IN PIPE</u>	<u>ELEVATION OF WATER IN PIPE</u>
1	46.0	216.6
2	34.1	204.1
3	44.7	219.3
4	29.6	207.9
5	14.0	194.6

The cover of the drainage manhole was removed and a reading obtained from the V notch weir. Water was observed to be flowing underneath the weir into the drainage pipe. The flowing water appeared clear and free of suspended solids. No seepage was observed at the toe or abutments of the embankment. In the vicinity of Station 6+50, approximately 82 feet south of the toe, a hole was located which was approximately 3 feet long, 12 inches wide, and 8 feet deep. There appeared to be 3 feet of water in the bottom of this feature.

To the east of the dam exists an emergency spillway channel which contains a "fuse plug." The fuse plug is approximately 90 feet long, 5 feet high, and 5 feet wide at the crest. The pavement appeared compact as a result of foot and vehicular traffic. The material excavated from a small test pit on the surface of the fuse plug was a gravelly silty coarse to fine sand.

c. Appurtenant Structures.

1) Inlet Spillway - The exterior was in excellent condition. The interior was in generally good condition with some spalling noted near the bottom. The base of the drop structure according to the plans is founded directly on bed-rock. The inspection of this feature indicated that an irregular scour hole approximately 5 feet deep has developed. Water is apparently flowing from this area into the outlet pipe at a steady rate.

2) Spillway Conduit - The interior of this 8 foot diameter conduit was inspected and found to be in generally good condition. Some hairline cracking was noted at the crown, with efflorescence and moisture also observed. The two sections of pipe nearest the inlet structure were found to have little or no mortar at the joints. No indication of spalling or deterioration of the concrete surfaces were noted. Alignment of the joints was good, there is an apparent slight sag in the pipe profile, about where the pipe passes beneath the dam crest.

3) Stilling Basin - The concrete endwall and wing-walls are in excellent condition. The bottom of the apron was not clearly visible due to the depth of water. The contacts between the embankment and the outlet structures were in good condition.

4) Blow-Off - The valve for the 30-inch blow-off pipe was easily opened by one man. The flow from the blow-off was clear and free of debris. Upon closing the valve, there was no leakage observed. The end of the blow-off pipe was in good condition.

d. Reservoir Area. The reservoir perimeter has well vegetated banks at moderate to steep slopes. There was no evidence of slides or sloughing. The depth of sediment and rate of accumulation in the reservoir is unknown.

e. Downstream Channel.

1) Spillway Channel (Mianus River) - The open channel downstream from the stilling basin was in excellent condition. It has large stone riprap forming stable banks, and there were no deposits of debris or sediment. The bottom and banks were nearly free of vegetation.

2) Wasteway Channel - This channel, excavated into earth and a limited area of rock, is overgrown with weeds and underbrush, approximately four feet high. Where visible, the sides and bottom were of stable earth with no evidence of erosion.

3.2 EVALUATION:

Based on the visual inspection, the dam appears to be in good condition. Trespassing has resulted in a loss of vegetation at several locations on the downstream slope and at the intersection with the left abutment from the toe to the crest of the downstream slope. Trespassing should be controlled, since it could lead to unacceptable long-term erosion. Brush and vegetation are growing at many locations on the riprapped slopes which could lead to development of erosion channel. For this reason, any vegetation in the riprap should be removed. Trespassing and vehicular traffic has resulted in compaction of soils contained in the fuse plug. Trespassing should be controlled, since it could lead to an unacceptable condition where the fuse plug would not erode as designed under high water.

The scoured area beneath the spillway inlet could lead to settlement of the spillway and outlet conduit.

The wasteway channel (downstream of fuse plug) is overgrown, reducing flow-carrying capacity.

SECTION 4 - OPERATIONAL PROCEDURES

4.1 PROCEDURES:

The Mianus Reservoir has a 2,450 million gallon capacity when full. It supplies water to Rockwood Lake and downstream to the Mianus Filter Plant. At Bargh there is a fuse plug for protection. When Bargh is full and a storm is coming, the blow-off (butterfly valve) is opened as additional protection.

4.2 MAINTENANCE OF DAM:

The dam is well maintained with a regular program of grass mowing and general maintenance in effect.

4.3 MAINTENANCE OF OPERATING FACILITIES:

The regulating gates and valves were tested and appear to be in mechanically good operating condition and are completely functional.

4.4 DESCRIPTION OF ANY WARNING SYSTEM IN EFFECT:

There was no warning system of any kind in effect at the time of the inspection. The Connecticut-American Waterworks Company is currently developing procedures which will provide for surveillance during peak flow conditions and a warning system.

4.5 EVALUATION:

The Mianus Reservoir Dam, which is over 20 years old, is very well operated and maintained. Although not designed for rapid drawdown, it should be noted that if the need should arise, drawdown could be effected only through the operational procedure of opening the 30-inch blow-off. Therefore, this valve should be periodically exercised to insure proper functioning.

SECTION 5 - HYDRAULICS/HYDROLOGY

5.1 EVALUATION OF FEATURES:

a. Design Data. There is no available information on the hydraulic design criteria for this dam and appurtenances. Under established criteria (OCE Guidelines) the recommended spillway design flood for the size (intermediate) and hazard potential (high) classification is the probable maximum flood (PMF). The PMF is the flood that may be expected from the most severe combination of critical meteorologic and hydrologic conditions that are reasonably possible in the region.

As estimate of the magnitude of the PMF at the site is based on an analysis of several sets of regional flood frequency data as presented in Appendix II.

As a conservative approach to the investigation, the more critical flood hydrograph was used. A peak inflow rate of 28,000 CFS was used as the test flood for determining spillway adequacy.

A stage-discharge relationship was calculated for both the spillway and the emergency wasteway. The calculations for the wasteway assumes that the fuse plug has been fully eroded to Elevation 252.

STAGE - DISCHARGE RATING

<u>Stage Elevation</u>	<u>Spillway</u>	<u>Q CFS Emergency Wasteway</u>	<u>Total</u>
252	0	0	0
253	120	270	390
254	330	760	1,090
255	550	1,400	1,950
256	720	2,160	2,880
257	840	3,020	3,860
258	940	3,970	4,910
259	1,040	5,000	6,040
260	1,090	6,110	7,200
261	1,160	7,290	8,450
262	1,220	8,540	9,760

The maximum spillway capacity, with no freeboard, is less than the peak discharge rate of the test flood (compare 9,760 CFS with 28,000 CFS). In order to determine the effect of the reservoir storage capacity, a hydrograph of the test flood was routed through the reservoir.

The hydrograph was formed by assuming the test flood had a duration of 24 hours, with the peak of 28,000 CFS occurring at 8 hours from the beginning of runoff. The rising and falling limbs of the hydrograph were assumed to be changing at a constant rate, forming a triangle. The routing operation indicated that the peak rate of discharge would not be significantly reduced, even if the fuse plug was assumed to be eroded to elevation 252 prior to the storm. The test flood routed through the reservoir reached elevation 264.9, 2.9 feet above the crest of the earth dam.

b. Experience Data. Discussion with water company personnel indicates that since the mid-1950's the dam has not been overtopped. However, flow has reached the crest elevation of the fuse plug.

c. Visual Observations. The on-site inspection of the dam and record information provided the data for the hydraulic evaluation of the spillways.

d. Overtopping Potential. The maximum spillway capacity is equal to 35 per cent of the spillway test flood peak flows. The peak rate of discharge from the test flood will overtop the embankment by about 2.9 feet.

SECTION 6 - STRUCTURAL STABILITY

6.1 EVALUATION OF STRUCTURAL STABILITY:

a. Visual Observations. There are no signs of structural instability evident.

b. Design and Construction Data. The design data presented in the previously noted references (see Section 2) do not point to any sources or areas of structural instability.

c. Operating Records. Five observation wells were installed in the downstream embankment early in 1957, see Appendix V. The water level readings made subsequent to the installation of these wells which have been provided indicate that the water level in the downstream shell is below that assumed for the stability analyses performed in the previously cited Haley and Aldrich reference, Reference d, Subsection 2.1. The available records suggest the free water surface is relatively unaffected by 11 foot variations in pool elevation. Given the assumption made for the friction angle, unit weights, and effective stresses that exist in this dam, it would appear that the factors of safety for normal seepage conditions (1.45) are close to what is usually required (1.5). Given the lower water levels recorded, there is no reason for concern about the factor of safety.

d. Post-Construction Changes. The five observation wells noted above were installed subsequent to construction of the dam. It is reported that the fuse plug has previously been replaced. However, details are uncertain.

e. Seismic Stability. This dam is in Seismic Zone 1 and, in accordance with recommended Phase I guidelines, does not warrant seismic analysis.

SECTION 7 - ASSESSMENT, RECOMMENDATIONS AND REMEDIAL MEASURES

7.1 DAM ASSESSMENT:

a. Condition. Based on the visual inspection, records available, and past operational performance, the dam is judged to be in good condition. However, there are some items which should be repaired to assure the continued safe performance and operation of the dam. The conditions that need to be remedied include trespassing on the downstream slope of the main dam and growth of brush on the riprapped upstream slope. In addition, trespassing on the crest of the fuse plug should be eliminated.

The project will not pass the test flood without overtopping the dam, and therefore, the total spillway capacity is inadequate. The spillway capacity is judged seriously inadequate since the project will not pass one-half the test flood without overtopping the dam (35 per cent).

b. Adequacy of Information. The information available was adequate for a Phase I inspection. In addition to visual inspection, design plans, specifications, and stability analyses were available. The evaluation of stability was limited to the strength data contained in the Woodward-Clyde-Sherard report. No data were available on the degree of density attained during construction.

c. Urgency. The recommendations and remedial measures should be implemented by the owner within 2 years after receipt of this Phase I Report.

d. Need for Additional Investigation. Additional investigation of the scour hole beneath the spillway base is necessary to determine methods of repair and should be undertaken by the owner.

7.2 RECOMMENDATIONS:

It is recommended that the following measures be undertaken by the owner:

1) The materials contained in the fuse plug should be excavated and replaced, without tamping.

2) The apparent leak underneath the weir, which is located within the drainage manhole, should be repaired. This would insure accurate measurement of the seepage from the toe drains by recording the volume of water being discharged.

3) Subsequent to studying feasible methods, the scour hole at the spillway should be repaired.

7.3 REMEDIAL MEASURES:

a. Alternatives. Not applicable.

b. Operation and Maintenance and Procedures.

1) The owner should control trespassing on the dam, particularly near the junction of the left abutment and the downstream slope where trespassing is extensive at the present time.

2) The brush and vegetation growth on the upstream riprapped surface should be controlled.

3) The hole, approximately 80 feet south of the toe of the dam in the vicinity of Station 6+50, should be properly backfilled.

4) The wasteway channel should be cleared of heavy brush and a dense stand of grass maintained.

5) Round the clock surveillance should be provided during periods of high precipitation.

6) The owner should develop a formal warning system. An operational procedure to follow in the event of an emergency should also be adopted.

7) The owner should provide continued periodic inspections at an annual frequency.

APPENDIX A

VISUAL INSPECTION - CHECK LIST

PERIODIC INSPECTION CHECK LIST

PROJECT Mianus Reservoir Dam

DATE August 3, 1978

INSPECTOR Richard F. Murdock

DISCIPLINE Geotechnical

INSPECTOR Robert C. Smith

DISCIPLINE Project Manager

AREA EVALUATED	CONDITION
<u>DAM EMBANKMENT</u>	
Crest Elevation	
Current Pool Elevation	
Maximum Impoundment to Date	
Surface Cracks	None observed
Pavement Condition	Crushed stone in good condition.
Movement or Settlement of Crest	None observed
Lateral Movement	None observed
Vertical Alignment	Good
Horizontal Alignment	Good
Condition at Abutment and at Concrete Structures	Good
Indications of Movement of Structural Items on Slopes	None observed
Trespassing on Slopes	Vehicular and foot paths at several locations on downstream slope.
Sloughing or Erosion of Slopes or Abutments	
Rock Slope Protection - Riprap Failures	Some local deterioration of riprap
Unusual Movement or Cracking at or near Toes	None observed
Unusual Embankment or Downstream Seepage	None observed

PERIODIC INSPECTION CHECK LIST

PROJECT Mianus Reservoir Dam

DATE August 3, 1978

INSPECTOR Richard F. Murdock

DISCIPLINE Geotechnical

INSPECTOR Robert C. Smith

DISCIPLINE Project Manager

AREA EVALUATED	CONDITION
<u>DAM EMBANKMENT</u> - (continued)	
Piping or Boils	None observed
Foundation Drainage Features	Drainage manhole collection basin weir needs repair
Toe Drains	Toe drains observed leading into manhole
Instrumentation System	

PERIODIC INSPECTION CHECK LIST

PROJECT Mianus Reservoir Dam

DATE August 3, 1978

INSPECTOR _____

DISCIPLINE _____

INSPECTOR _____

DISCIPLINE _____

AREA EVALUATED	CONDITION
<p><u>OUTLET WORKS - OUTLET STRUCTURE AND OUTLET CHANNEL</u></p> <p>General Condition of Concrete</p> <p>Rust or Staining</p> <p>Spalling</p> <p>Erosion or Cavitation</p> <p>Visible Reinforcing</p> <p>Any Seepage or Efflorescence</p> <p>Condition at Joints</p> <p>Drain Holes</p> <p>Channel</p> <p>Loose Rock or Trees Over- hanging Channel</p> <p>Condition or Discharge Channel</p>	<p>Not Applicable</p>

PERIODIC INSPECTION CHECK LIST

PROJECT Mianus Reservoir Dam

DATE August 3, 1978

INSPECTOR _____

DISCIPLINE _____

INSPECTOR _____

DISCIPLINE _____

AREA EVALUATED	CONDITION
OUTLET WORKS - CONTROL TOWER	None
a. Concrete and Structural	
General Condition	
Condition of Joints	
Spalling	
Visible Reinforcing	
Rusting or Staining of Concrete	
Any Seepage or Efflorescence	
Joint Alignment	
Unusual Seepage or Leaks in Gate Chamber	
Cracks	
Rusting or Corrosion of Steel	
b. Mechanical and Electrical	
Air Vents	
Float Wells	
Crane Hoist	
Elevator	
Hydraulic System	

PERIODIC INSPECTION CHECK LIST

PROJECT Mianus Reservoir Dam

DATE August 3, 1978

INSPECTOR _____

DISCIPLINE _____

INSPECTOR _____

DISCIPLINE _____

AREA EVALUATED	CONDITION
<p><u>OUTLET WORKS - CONTROL TOWER</u> (continued)</p> <p>Service Gates</p> <p>Emergency Gates</p> <p>Lightning Protection System</p> <p>Emergency Power System</p> <p>Wiring and Lighting System In Gate Chamber</p>	<p>None</p>

PERIODIC INSPECTION CHECK LIST

PROJECT Mianus Reservoir Dam

DATE August 3, 1978

INSPECTOR James MacBroom

DISCIPLINE Hydraulics/
Hydrology

INSPECTOR Robert C. Smith

DISCIPLINE Project Manager

AREA EVALUATED	CONDITION
<u>OUTLET WORKS - SPILLWAY WEIR, APPROACH AND DISCHARGE CHANNELS</u>	Wasteway
a. Approach Channel	
General Condition	Extensive vegetation along bottom of channel.
Loose Rock Overhanging Channel	
Trees Overhanging Channel	None
Floor of Approach Channel	See above
b. Weir and Training Walls	
General Condition of Concrete	Upstream and downstream portions separated by fuse plug; surface has been compacted due to trespassing.
Rust or Staining	
Spalling	
Any Visible Reinforcing	
Any Seepage or Efflorescence	
Drain Holes	
c. Discharge Channel	
General Condition	Poor
Loose Rock Overhanging Channel	None
Trees Overhanging Channel	Trees adjacent to channel, extensive vegetation.
Floor of Channel	
Other Obstructions	

PERIODIC INSPECTION CHECK LIST

PROJECT Mianus Reservoir Dam

DATE August 3, 1978

INSPECTOR James MacBroom

DISCIPLINE Hydraulics/
Hydrology

INSPECTOR Robert Smith

DISCIPLINE Project Manager

AREA EVALUATED	CONDITION
<u>OUTLET WORKS - TRANSITION AND CONDUIT</u>	Spillway
General Condition of Concrete	Good
Rust or Staining on Concrete	None
Spalling	
Erosion or Cavitation	Deep scour hole at base of spillway. Hairline cracking..
Cracking	
Alignment of Monoliths	
Alignment of Joints	Good
Numbering of Monoliths	

PERIODIC INSPECTION CHECK LIST

PROJECT Mianus Reservoir Dam

DATE August 3, 1978

INSPECTOR _____

DISCIPLINE _____

INSPECTOR _____

DISCIPLINE _____

AREA EVALUATED	CONDITION
<p><u>OUTLET WORKS - SERVICE BRIDGE</u></p> <p>a. Super Structure</p> <p> Bearings</p> <p> Anchor Bolts</p> <p> Bridge Seat</p> <p> Longitudinal Members</p> <p> Under Side of Deck</p> <p> Secondary Bracing</p> <p> Deck</p> <p> Drainage System</p> <p> Railings</p> <p> Expansion Joints</p> <p> Paint</p> <p>b. Abutments & Piers</p> <p> General Condition of Concrete</p> <p> Alignment of Abutment</p> <p> Approach to Bridge</p> <p> Condition of Seat & Backwall</p>	<p>None</p>

APPENDIX B
ENGINEERING DATA

CHECK LIST
 ENGINEERING DATA
 DESIGN, CONSTRUCTION, OPERATION
 PHASE I

NAME OF DAM Mianus Reservoir
 I.D. NO. CT 00050

ITEM	REMARKS
AS-BUILT DRAWINGS	None Exist
REGIONAL VICINITY MAP	Available From U.S.G.S.
CONSTRUCTION HISTORY	Unknown
TYPICAL SECTIONS OF DAM	Available From Plan
OUTLETS - Plan	From Plans
- Details	From Plans
- Constraints	Unknown
- Discharge Ratings	Unavailable
RAINFALL/RESERVOIR RECORDS	From Connecticut-American Waterworks Co.
DESIGN REPORTS	From Connecticut-American Waterworks Co.
GEOLOGY REPORTS	None
DESIGN COMPUTATIONS	None
HYDROLOGY & HYDRAULICS	None
DAM STABILITY	From Connecticut-American Waterworks Co.
SEEPAGE STUDIES	From State D.E.P. Records
MATERIALS INVESTIGATIONS	None
BORINGS RECORDS	On Plans
LABORATORY	From Reports/Studies
FIELD	None

CHECK LIST
ENGINEERING DATA
DESIGN, CONSTRUCTION, OPERATION
PHASE I

NAME OF DAM Mianus Reservoir

I.D. NO. CT 00050

ITEM	REMARKS
POST-CONSTRUCTION SURVEYS OF DAM	
BORROW SOURCES	None
MONITORING SYSTEMS	Unknown
MODIFICATIONS	Yes, Water Levels, Seepage Quantity
HIGH POOL RECORDS	Monitoring System
POST-CONSTRUCTION ENGINEERING STUDIES AND REPORTS	From Connecticut-American Waterworks Co.
PRIOR ACCIDENTS OR FAILURE OF DAM DESCRIPTION REPORTS	From State D.E.P.
MAINTENANCE OPERATION RECORDS	None
SPILLWAY PLAN	From Connecticut-American Waterworks Co.
SECTIONS	Plans
DETAILS	Plans
OPERATING EQUIPMENT PLANS & DETAILS	Plans Plans

REPORT ON
STABILITY AND SEEPAGE ANALYSES
MIANUS RIVER DAM
STAMFORD, CONNECTICUT
APRIL 1959

for
Water Resources Commission
State of Connecticut

by
Haley & Aldrich
Consulting Soil Engineers
Cambridge, Mass.

HALEY & ALDRICH
CONSULTING SOIL ENGINEERS
238 MAIN STREET, CAMBRIDGE 42, MASS.
UNIVERSITY 4-2779

April 30, 1959

Mr. John J. Mozzochi
Consulting Engineer
265 Hebron Avenue
Glastonbury, Connecticut

Subject: Report on Stability and Seepage Analyses
Mianus River Dam, Stamford, Connecticut

Dear Mr. Mozzochi:

Submitted herewith is our report of stability and seepage investigations in connection with the Mianus River Dam. This study was made in accordance with the first phase of our contract, dated 2 January 1959, with the Water Resources Commission, State of Connecticut. The study was based on a review of pertinent correspondence, records, the design study report, contract plans and specifications.

The dam site was visited and information on as-built conditions were obtained in discussions with Mr. David M. Greer, Greer Engineering Associates and by review of daily reports of the soils control engineer.

After you and personnel of the Commission have had an opportunity to review the report we would be glad to discuss or furnish any further details desired. If you feel that additional assurance as to the stability of the dam is desirable, then the program of field observations and tests contained in the recommendations could be initiated.

We appreciate the opportunity of working with you on this project and will be pleased to be of further service.

Very truly yours,
HALEY & ALDRICH

Steve J. Poulos
S. J. Poulos

J. F. Haley
J. F. Haley

Submitted: 3 copies
Water Resources Commission, 3 copies

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REFERENCES

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<u>No.</u>	<u>Title</u>
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2.	Stability Analysis
3.	Plot of Previous Field Measurements
4.	Location Plan of Field Investigations
5.	Displacement Stakes and Wellpoint Installation
6.	Trial Circles for Stability Analysis

SYNOPSIS

The purpose of this report is to determine the estimated stability of the downstream slope of the Mianus River Dam and the cause of the high seepage losses which have been measured.

A flow net obtained graphically was modified in accordance with data measured in the field and used as a basis for seepage loss and seepage force computations. The estimated seepage losses came to approximately 275,000 gallons per day using what are believed to be reasonable permeability assumptions. The measured seepage loss, after subtracting surface runoff, is approximately 570,000 gallons per day. The agreement between the two figures is believed to be reasonable when it is considered that variations in permeability by factors of 10 and even greater are common in stream bed deposits.

Stability analyses which included the effect of seepage forces resulted in an estimated factor of safety of 1.45 for the case of zero cohesion and an angle of internal friction of 35 degrees in the embankment and foundation soils. This factor of safety is considered to be adequate for the steady seepage state with maximum reservoir level. Inclusion of horizontal earthquake forces equal to 0.1 times the acceleration of gravity produces a factor of safety of 1.05, which is considered satisfactory for the extreme assumptions made.

Assumptions that are in our opinion reasonably conservative for the soil properties would lead to the conclusion that the dam is safe as constructed. However, in order to arrive at a more definite indication of the margin of safety of the downstream slope of the dam it would be necessary to obtain additional information on soil properties and seepage conditions in the dam. It is therefore recommended if more positive assurance than was possible from this study is desired, that a program of field and laboratory investigations be undertaken to measure the required soil properties to confirm or refine the assumptions used in the analysis.

I. INTRODUCTION

1-01. GENERAL

This report of studies of seepage and stability of the Mianus River Dam, Stamford, Connecticut, is submitted in fulfillment of the first phase of the contract between the Water Resources Commission and Haley & Aldrich, entitled Consulting Engineer Services for Inspection and Reports on Dams, Dikes, Reservoirs and Similar Structures dated 2 January 1959. Measurements of flow through the dam have previously been made which indicate that approximately six times the original estimate of seepage loss has occurred. Standpipes installed in the downstream shell of the dam show that the piezometric level to be relatively high. Measurements of the surface elevation of the downstream slope at various locations along the dam show that the actual shape of the slope differs from the design. The possibility that movement has occurred since completion of construction was therefore postulated.

1-02. PURPOSE

The purpose of this study is to determine the indicated stability of the downstream slope of the Mianus River Dam and the effect on the stability of the measured piezometric level and the seepage losses. It is also the purpose of this report to formulate a suggested program of field and laboratory investigations so that a more thorough analysis of the stability of the dam may be made in the event that available data and analyses are found to be insufficient for the purposes of the Commission.

1-03. SCOPE

The scope of this report includes the review of past correspondence concerning the stability of the downstream slope of the Mianus River Dam and an analysis of its stability based upon the information contained therein. Data on previous field measurements of seepage loss through the dam, shape of the downstream slope and water surface elevations were made

available to this office by the Water Resources Commission. No field studies were made directly in connection with this report. We have made no attempt to confirm the hydrologic study or to determine the stability of the upstream slope.

The dam was examined by Mr. James F. Haley of this firm on January 27, 1959 and again on January 30, 1959. On the latter date Mr. John J. Curry, Chief Engineer and Mr. John J. Mozzochi, Consultant for the Water Resources Commission were present during the inspection. On January 29, 1959 Mr. Haley visited the office of Greer Engineering Associates, Montclair, New Jersey, and discussed the design and construction of the dam with Mr. David M. Greer of that firm. Mr. Greer made available for review results of their studies and also the soil control reports made during construction. These reports contained the results of field density and permeability tests and grain size distribution of the materials used in the various portions of the construction. The shear strength and permeability characteristics used in the analyses in this report were based or assumed primarily from the review of the soil control reports.

II. SEEPAGE ANALYSES

2-01. SEEPAGE LOSS

The flow net for a section of the dam at Station 5+0 is shown in Figure 1. In sketching the net, it was assumed, based on field control reports, that the upstream shell and core were of the same permeability and that it was one-tenth that of the downstream shell. For the purpose of drawing the flow net the downstream shell and foundation were assumed to have equal permeabilities and the stratification ratio in all parts of the dam was assumed to be 9. Since the upstream portion of the embankment is relatively impermeable when compared with the foundation, seepage loss estimates may be made by considering the effects of the foundation and embankment individually without substantial error. This was done in all of the following analyses.

In comparing the measured manhole outflow with estimates from the flow net it is necessary to make proper assumptions for permeability and to discount flows due to rainfall and subsurface seepage that by-passes downstream drainage system. Available information shows that the core and upstream shell have a permeability between 100 and 500 feet per year. The foundation permeability varies between 3100 and 5400 feet per year. The permeability of the downstream shell is in the vicinity of 1800 feet per year. Permeabilities referred to are in a horizontal direction. The assumed values of permeability for the purpose of computing quantity of flow are:

Core and upstream shell . . .	180 ft/yr.
Downstream shell	1800 ft/yr.
Foundation	3600 ft/yr.

As shown in the flow net, not all of the seepage loss is reflected at the weir which measures flow out of the manhole in the rock toe. Thus approximately 1.1 flow paths in the foundation must be discounted when comparing estimated and measured flows. Also, flow measured at the manhole reflects surface runoff after a rainfall. This fact is vividly demonstrated in the lower curves

of Figure 3 which compare weekly rainfall with weekly measurements of manhole outflow. Curve (a) shown connecting the low points of the plot of manhole outflow is the best estimate of measured seepage loss through the dam. The remainder of the flow should be credited to the surface infiltration of rainfall.

Calculations based upon the values of permeability assumed above, the flow net in Figure 1 and a pool elevation of 253, with proper allowance for the location of the measuring weir, result in an estimated seepage loss of 275,000 gallons per day. From curve (a) in Figure 3 at a pool elevation of 253, a reasonable estimate for measured seepage loss is 570,000 gallons per day. Permeability variations from field or laboratory test results by factors of 10 or even greater in deposits, such as those considered herein, are common. We therefore feel that the agreement between the estimated and the measured losses is entirely reasonable. The quantity of flow in itself does not affect stability and need only be considered from the standpoint of function for which the dam was constructed. The various factors that do affect stability are discussed in Part III.

In the event that the pool elevation rises to 260 and remains there long enough to affect seepage loss, the flow measured at the manhole may be expected to increase by 100,000 gallons per day over the amount of flow measured with the pool at elevation 253, as shown in Figure 3.

To compute the estimated seepage losses it was necessary to use an "effective length" for the foundation and core. The "effective length" is that length of dam of constant cross section equal in geometry and scale to that shown in Figure 1 which allows seepage loss equal to the actual dam with its varying geometry and scale. The effective length of the foundation was obtained by means of a weighted average using the relationship between flow per unit head drop and the ratio of base width to foundation thickness for flow beneath impermeable dams. This relationship is plotted on page 212

of reference (1). An average value of flow per unit head loss over the length of the dam was computed by inserting the actual ratios of base width to foundation thickness into the above referenced graph and obtaining the corresponding flow per unit head loss. Dividing this average by the value for the section on which the flow net was drawn and multiplying by the length of the actual dam results in the new effective length of foundation. This method yields a value of 262 feet, which agrees with that used in the original design study. The equivalent length of the core was similarly obtained by use of the relation that seepage through the core is proportional to the quantity:

$$(d^2 + h_t^2)^{1/2} - d$$

where "d" is a horizontal distance which depends upon the cross section of the dam and "h_T" is the total head loss through the dam. This relationship, given on page 178 of reference (2), results in an effective length of core of 610 feet. Both of the above methods are approximate and based upon the assumption previously stated that flow beneath and through the upstream portion of the dam may be considered independently without causing serious error.

2-02. FACTORS AFFECTING SEEPAGE LOSS

Seepage loss through and beneath the dam is dependent upon permeability, boundary conditions and headwater elevation. The measured seepage loss is affected by the surface runoff and the location of the measuring weir as noted above.

a. Permeability: The sensitivity of seepage losses to the permeability of the soils is shown by the plot contained in Figure 1. The formula for computing flow which is also shown on Figure 1 indicates that flow is directly proportioned to permeability.

The ratios of permeabilities between the various portions of the embankment and the foundation also affect the flow since changing ratios changes the flow net. The ratio of the number of flow paths to the number of equipotentials would therefore change in the formula used for computing

flows.

The stratification ratio also affects flow by changing the flow net. Higher stratification ratios result in higher rates of flow if the effective permeability

$$(k_v \times k_h)^{1/2}$$

remains constant.

Of the three effects of permeability mentioned above, it is the absolute values which most strongly affect flow. It is therefore necessary to obtain more accurate values before refinements in seepage computations can be made. The ratios of permeability do not strongly affect quantity of flow, however they do have substantial effect on seepage forces and stability as discussed in Part III.

b. Boundary Conditions: The boundary conditions which are variable and affect the rate of flow appreciably are the back pressures in the graded filter and rock toe. If the porous pipe in the filter is plugged, or if the filter does not operate properly, the pressure within it will rise and cause a decrease in the rate of flow through and beneath the dam. This also holds true for a rise in pressure in the rock toe. A more important effect of this increased pressure in the filters is its affect on stability as shown in Part 3-02. Rough computations of the capacity of the porous concrete drains to pass the seepage water through its walls indicate that no back pressure will result from the use of the pipe as shown in the plans. A filter constructed of the materials and in the manner as proposed in the design would, in our opinion, result in satisfactory filter operation.

Another boundary effect is the three dimensional variation in the foundation thickness and dam height. Consideration was given to this effect by computing the effective lengths mentioned above, however each cross section was analysed independently of the adjacent section. In reality,

the three dimensional effect causes the flow through each section to be affected by its neighbor i. e., a component of the flow in a direction perpendicular to the main river direction does occur in the dam. These lateral velocities are likely to be small in the central portions of the dam and would tend to increase at the edges where the ground and bedrock slopes are steeper. Since the majority of the flow occurs through the central portions, the effect on flow rates of three dimensional lateral velocities should be small. The three dimensional effect also explains the fact that the measured free water surface elevation is higher near the abutments of the dam. Since the ground elevation and gravel filter are at a higher elevation near the abutments and the pool elevation is constant across the dam, the piezometric surface does not have to drop as rapidly to meet the tailwater elevation.

c. Headwater Elevation: Assuming the flows through the embankment and foundation to be independent of each other, the flow through the foundation is directly proportional to headwater elevation. The flow net in the embankment changes as headwater elevation changes. Therefore the values of n_f and n_d as well as the total head must be changed when computing the flow through the embankment. In computing the values given above for seepage losses with a pool elevation of 253, an estimate of the change in n_f was made. Since the change is small, little error is introduced. The change in n_d in this case is smaller and was neglected.

d. Surface Runoff: An indication of the quantity of surface runoff reaching the manhole can be obtained only by considering the duration and rate of precipitation in any storm, the drainage area and the runoff relationships. A rough calculation shows that peak flows of one to 1.5 million gallons per day can be expected from a storm in which 4" of rainfall occurs in 8 hours if the runoff is 60% of 70% of the precipitation. These are reasonable assumptions and the quantity checks with the peak flow of 2.02 million gallons per day noted in Figure 3. This latter value includes seepage losses through and beneath the dam on the order of 0.6 million gallons per day.

III. STABILITY ANALYSES

3-01. RESULTS

Stability analyses using the method shown in Figure 2 on the trial circles shown in Figure 6 result in a factor of safety of 1.45, assuming that there is no cohesion in the embankment or foundation and that the friction angle is constant at 35° . These assumptions of soil strength are considered conservative. Inclusion of earthquake forces equal to one-tenth the acceleration of gravity in the horizontal direction decreases the safety factor to 1.05. Earthquake forces are not generally considered in stability studies for earth dams in New England, however, we have included this possibility in our study because of the downstream occupancy. The low factor of safety for this extreme loading condition is considered satisfactory. In all stability calculations, the seepage forces caused by flow through the dam were included. To obtain realistic values of these forces, the free surface of flow was modified by field measurements as shown in the notes of Figure 1.

Based on these preliminary studies, the downstream slope thus appears to be safe. To obtain a more refined estimate of the safety of the downstream slope it would be necessary to make field and laboratory investigations to determine the soil properties more accurately to confirm the assumptions. The effect of each of the assumed soil properties on the stability is discussed in the following section.

3-02. FACTORS AFFECTING STABILITY

a. Friction Angle: As shown in the graph contained in Figure 2 increasing the friction angle markedly increases the factor of safety. The normal range of friction angles for soils of the type in the downstream portion of the embankment and the foundation is between 33 and 45 degrees, with the higher values being more prevalent.

b. Cohesion: Since the cohesive force resisting failure is the product of arc length and cohesion, the factor of safety increases with increasing

cohesion. This relation is also evident from the graph in Figure 2. The rate of change of the factor of safety with the value of cohesion is greater for trial circles entirely in the embankment portion of the dam. This is true since a greater proportion of the total force resisting failure is in the cohesive strength for those circles. The foundation soils probably have low cohesion while the downstream materials would have only slight cohesion.

c. Seepage Forces: Seepage forces caused by the friction between soil and water while water is flowing through the dam result in lower safety factors than occur in identical dry embankments. Seepage forces are increased in the downstream slope and stability is therefore decreased when a) the ratio of the permeability of the downstream shell to the core or the foundation is decreased, b) the ratio of horizontal to vertical permeability is increased and c) back pressures build up in the graded filter or rock toe. All three results in a rise in the free water surface within the downstream portion of the dam.

The above factors indicate that accurate measurements of the permeability ratios involved and the pressure in the graded filter are required to adequately analyze the embankment. If field investigations indicate that the rock toe is clogged, it is also advisable to measure the pressure therein.

d. Unit Weight: The unit weight of soil in the embankment also affects the stability. Except in the case of zero cohesion, an increase in unit weight will cause a small decrease in safety factor if other variables are held constant. Unit weight has an even smaller effect on the safety factor when no cohesion exists.

e. Earthquakes: Greenwich, Connecticut is generally considered to be in a seismically active area although no earthquakes of consequence have occurred there in the recent past. For the design of earth dams earthquake forces acting horizontally on the failure mass with a magnitude of 0.1 times its weight are sometimes used in such seismic regions. Under these conditions a safety factor of just over one is deemed adequate since

the possibility that maximum seepage and earthquake forces will act concurrently is rare. There are few if any well documented failures of earth dam embankments that have been credited to earthquake forces. On the other hand, several instances of earth dams that have withstood earthquakes may be found in the literature.

IV. CONCLUSIONS

Based on the stability and seepage analyses reported herein, made using what are believed to be conservative assumptions of shear strength and permeability of foundation and embankment soils, the conclusions are summarized as follows:

1. The indicated minimum factor of safety of the downstream portion of the dam with the reservoir at maximum level and a steady seepage state is 1.45. This factor of safety is lowered to 1.05 when earthquake forces are considered. Factors of safety of these magnitudes are considered adequate for the assumed conditions. The indicated stability of the dam would decrease if the free water surface in the downstream portion of the dam rises to a higher level than assumed in the analyses or if the foundation and embankment soils have less shear strength than assumed.
2. The stability study indicates that the factor of safety against a relatively shallow slide within the downstream slope is 1.61. Thus it is believed that the apparent sloughing at the downstream toe is due to erosion of the topsoil or material from the slope surface prior to establishment of turf cover and possibly shallow creep of the surface materials occurring during frost melting periods.
3. The measured quantity of seepage flowing through the embankment and foundation into the drainage system is approximately 0.88 cu. ft. per sec as compared with an estimated quantity from flow net analysis of 0.42 cu. ft. per sec. These figures are in reasonable agreement considering the possible variations in permeability of the soils and the assumptions of boundary conditions that are necessary in the flow net analyses.
4. In order to make more refined analyses of the stability it will be necessary to perform the field and laboratory test program outlined in Part V.

V. RECOMMENDATIONS

5-01. FIELD INVESTIGATIONS

A layout of the recommended field test program is shown in Figure 4, if it is desired to obtain a more definitive estimate of safety of the downstream portion of the dam.

a. Test Pits: Six test pits approximately three feet square and five feet deep should be dug where shown. Sufficient samples are to be taken to provide for the laboratory tests outlined in paragraph 5-02. Test Pit No. 5 is to penetrate through the soil cover and into the rock toe so that a visual indication of whether or not fines have migrated into the toe may be obtained. Pit No. 6 is to be excavated to determine the properties of the foundation since the critical failure circle shown in Figure 2 is partially in the foundation material. The other four pits are to be dug at varying elevations in the embankment in an attempt to reflect the properties of material from various borrow pits used during construction.

In the process of excavation or at adjacent locations, field densities of the material to be laboratory tested should be taken.

b. Water Observations: One wellpoint and two standpipes should be installed as shown. The standpipes will reveal the location of the free surface near the toe of the dam where its elevation is very sensitive to changes in permeability of the soil in the embankment and is difficult to establish analytically. These standpipes should be driven to the elevation shown or as governed by field conditions. A minimum depth, just sufficient to reach the free surface, should be drilled.

The wellpoint is to be installed in the graded filter in order to determine the pressure therein. As drilling proceeds, the change in material from the downstream shell to the filter should be evident from the washings and an elevation measurement. The wellpoint itself (see sketch in Figure 5) should be imbedded in the filter gravel.

c. Displacement Stakes: Lateral, or downward movement of the downstream slope may be detected from continual observation of the displacement stakes shown in Figure 4. A typical stake is shown in Figure 5.

d. Other: In addition to the above field measurements, a limited number of manhole outflow, rainfall and pool elevation records should be made to obtain a more complete correlation of data, and to determine if any changes in seepage quantities have occurred since the time of previous observations.

5-02. LABORATORY TESTS

Using samples obtained from the test pits, the following laboratory tests are recommended:

- a. Triaxial tests to determine shear strength and permeability.
- b. Grain size analyses.
- c. Permeability tests separate from the triaxial test to obtain the variation of permeability with void ratio.

Standard classifications, water content, Atterberg limits (if applicable) and specific gravity measurements should also be made to supply the information required for analysis.

5-03. ANALYSIS

The primary object of the analysis is to obtain proper seepage forces and soil properties to evaluate stability. A flow net corrected on the basis of measured water surface elevations, filter pressures and permeabilities should first be drawn. If there is no appreciable change in seepage forces from those used herein, then it is only necessary to insert the new shear strength data into the stability trials already completed. Any substantial change in seepage forces will necessitate a stability analysis using the newly measured values of cohesion, friction, unit weight and seepage forces.

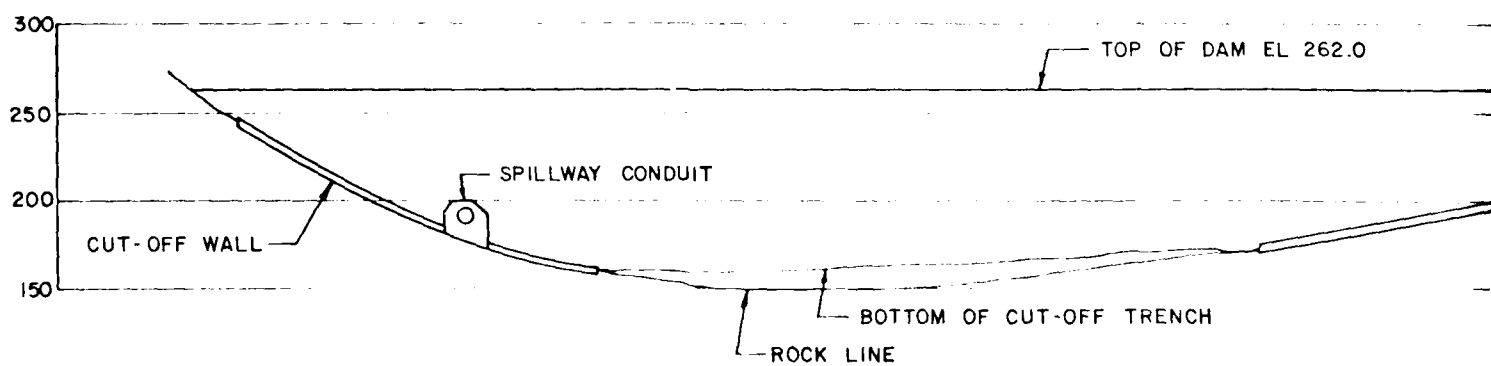
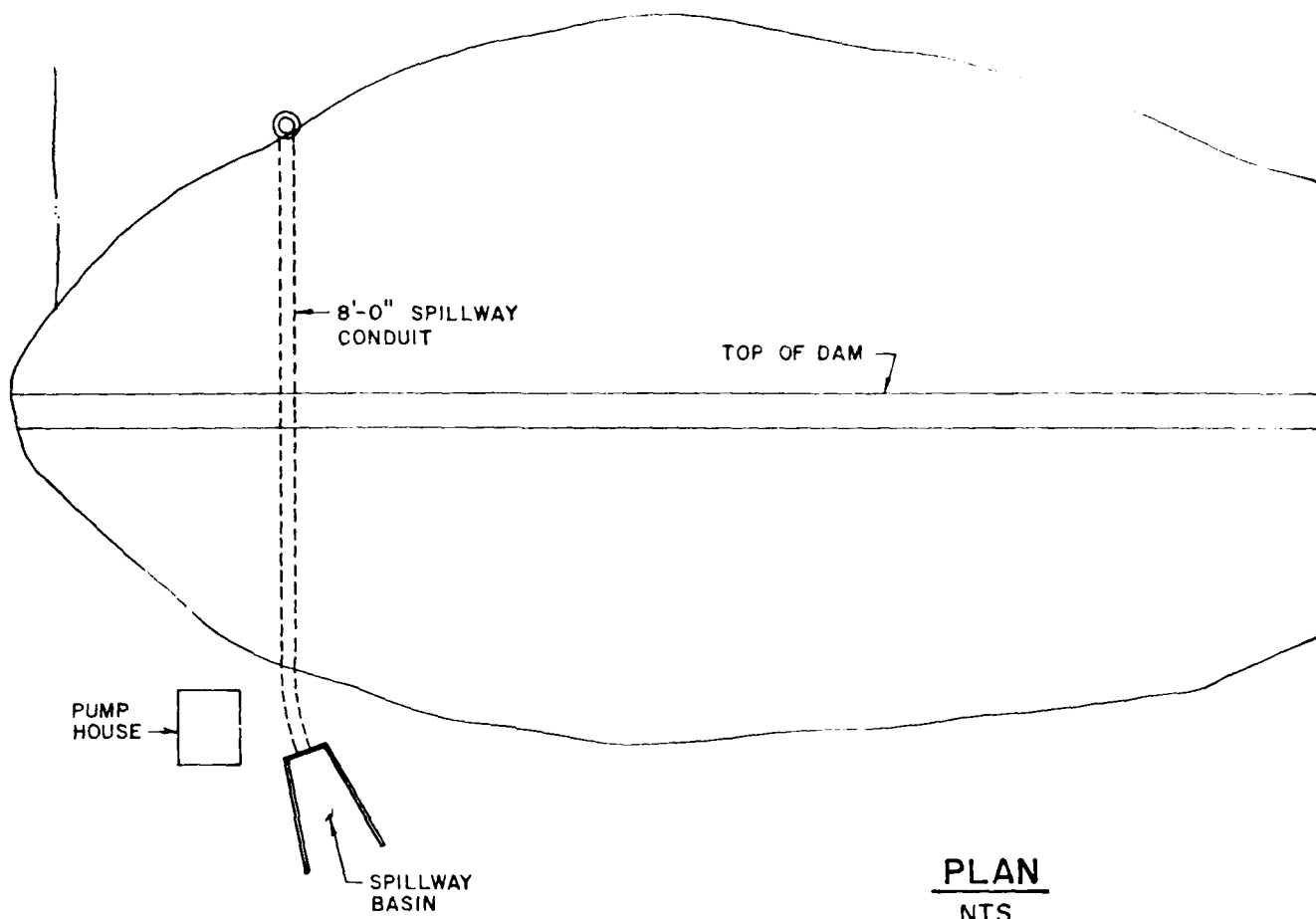
It is possible that the present variation of the downstream slope was caused by erosion prior to establishment of the turf cover. If this is true the grain size analyses of the upper strata of soils near the toe of the slope,

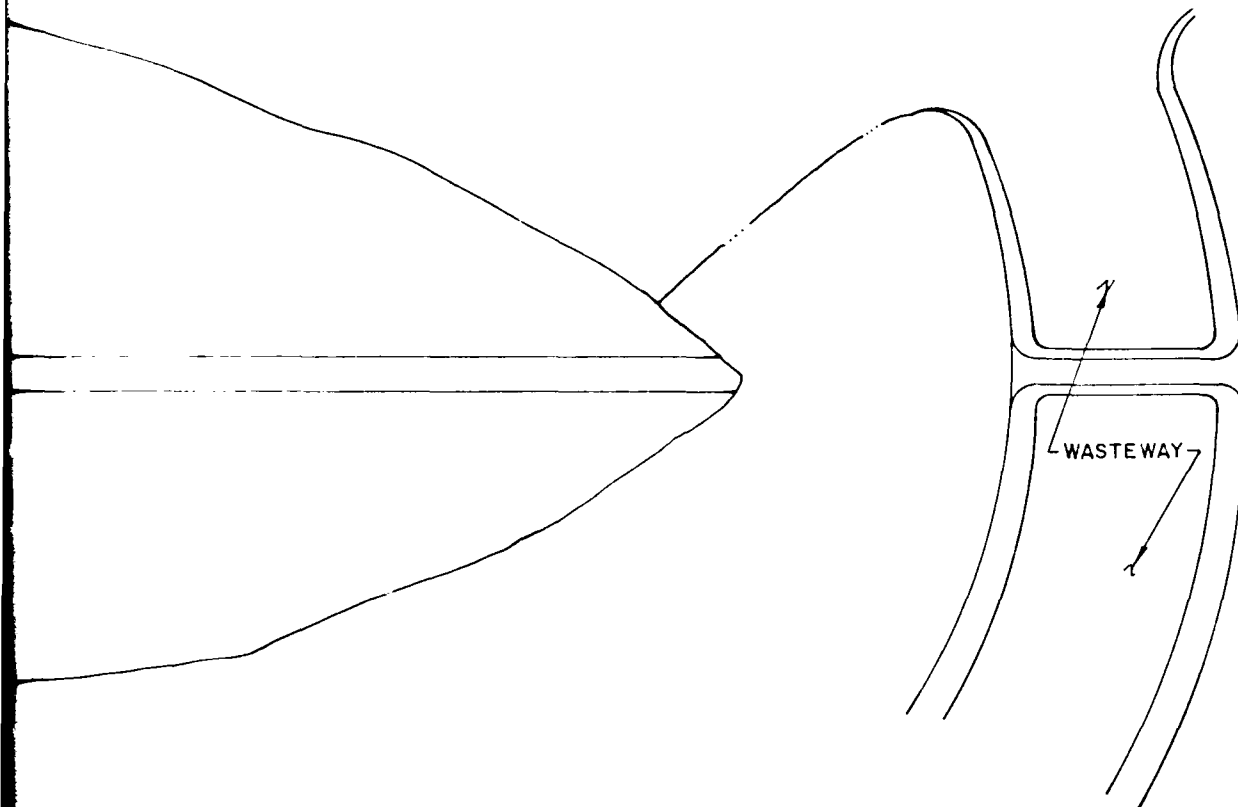
will indicate that they are the finer components of the soils located higher on the slope which have been washed down.

The above combined with the results of displacement stake observations will permit proper evaluation of the stability of the downstream slope. It should be pointed out that the above recommended tests may substantiate assumptions made herein and obviate the requirement for involved calculations over and above those completed to date.

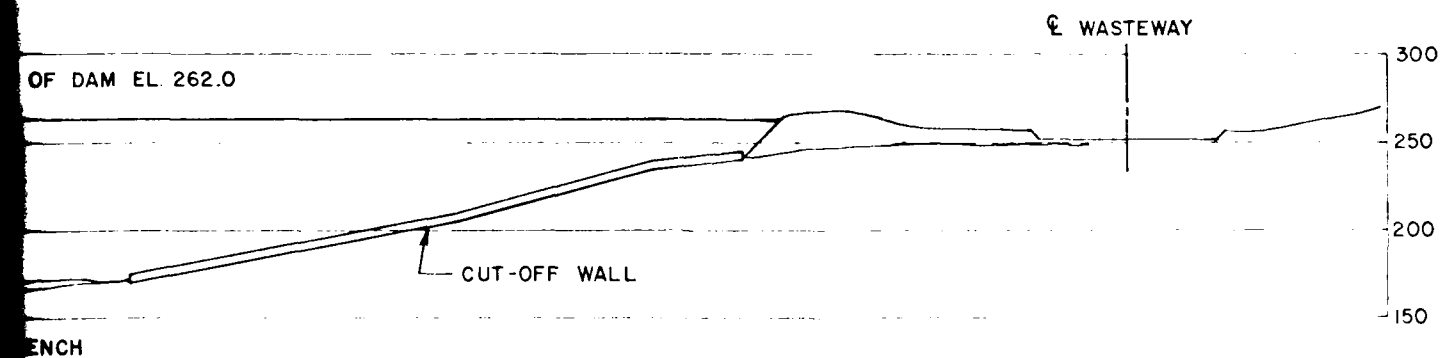
REFERENCES

- (1) Muskat, M., "The Flow of Homogeneous Fluids through Porous Media", New York, McGraw Hills, 1937.
- (2) Taylor, D. W., "Fundamentals of Soil Mechanics," New York, Wiley 1948
- (3) Greer and McClelland, "Investigation and Design of the Mianus River Dam, Stamford, Connecticut", November 1954.



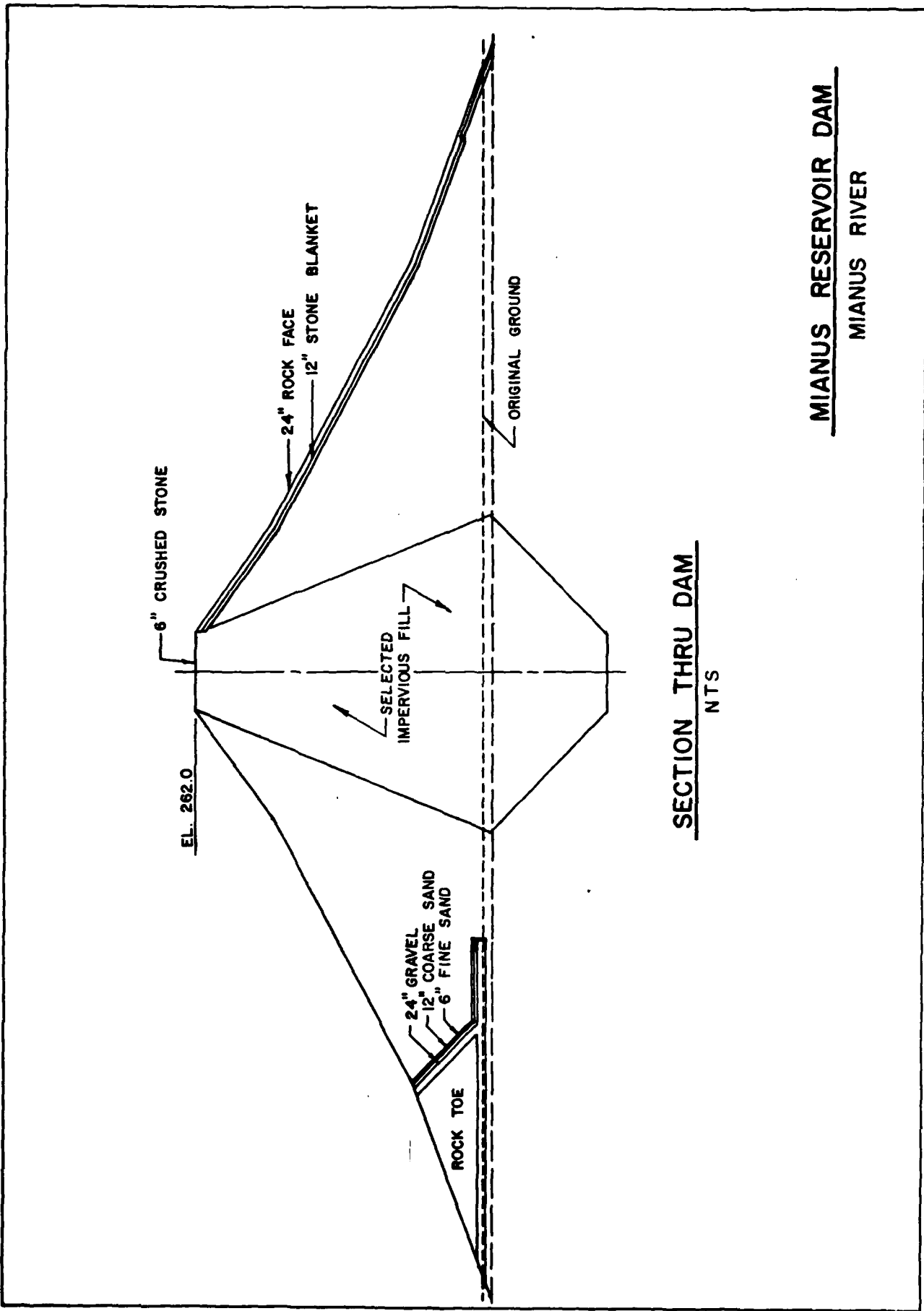


PLAN
NTS



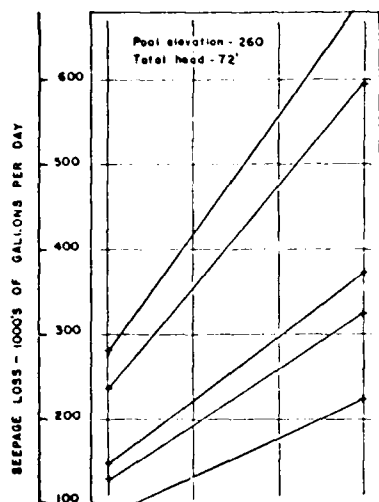
SECTION OF DAM
NTS

MIANUS RESERVOIR DAM
MIANUS RIVER



SECTION THRU DAM
NTS

MIANUS RESERVOIR DAM
MIANUS RIVER



Total seepage (Q)

Total seepage to manhole

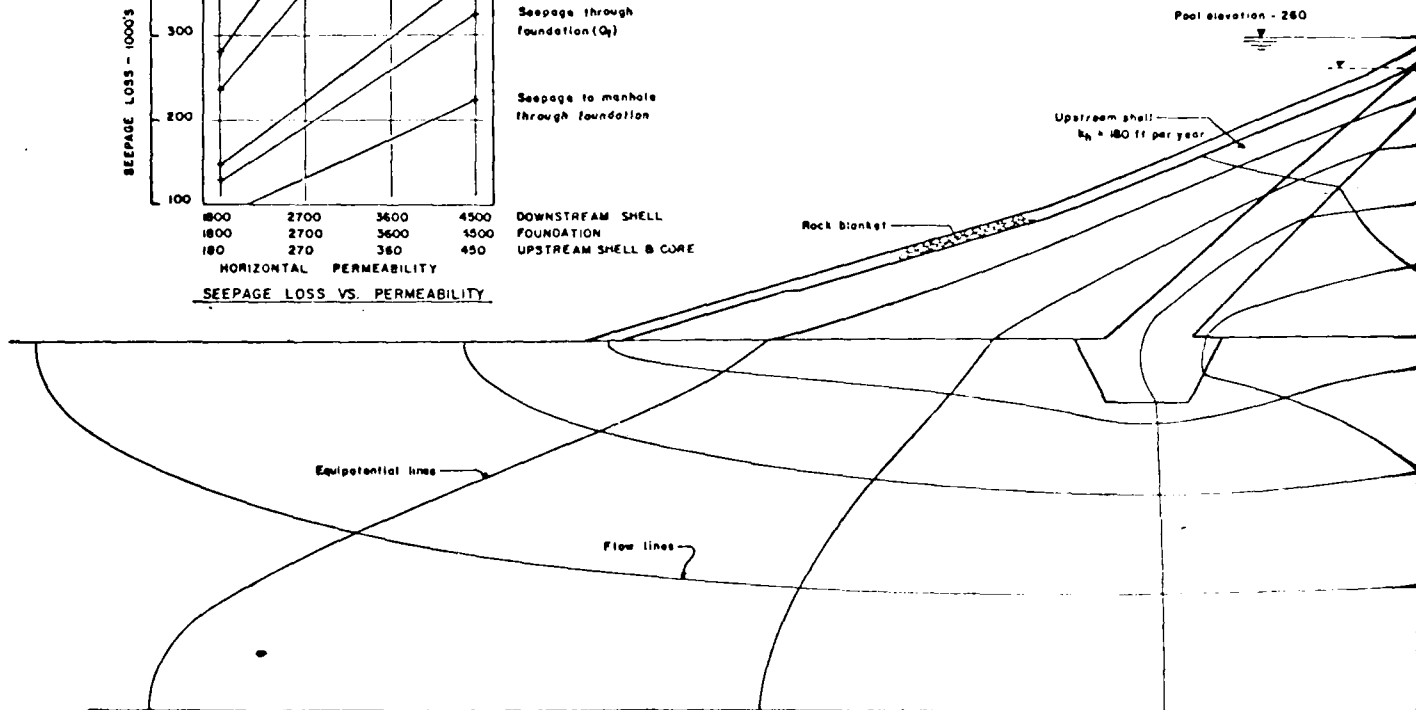
Seepage through embankment (Q_e)

Seepage through foundation (Q_f)

Seepage to manhole through foundation

DOWNSTREAM SHELL
FOUNDATION
UPSTREAM SHELL & CORE

SEEPAGE LOSS VS. PERMEABILITY



FLOW NET AT CROSS

NOTATION

- k_v = Vertical coefficient of permeability (ft./yr.)
 k_h = Horizontal coefficient of permeability (ft./yr.)
 n_d = Number of equipotential drops
 n_f = Number of flow paths
 H = Total head (ft.)
 Q = Rate of seepage loss through dam (gal/day)
 L_e = Effective length of embankment (ft.)
 L_f = Effective length of foundation (ft.)
 k_h/k_v = Stratification ratio

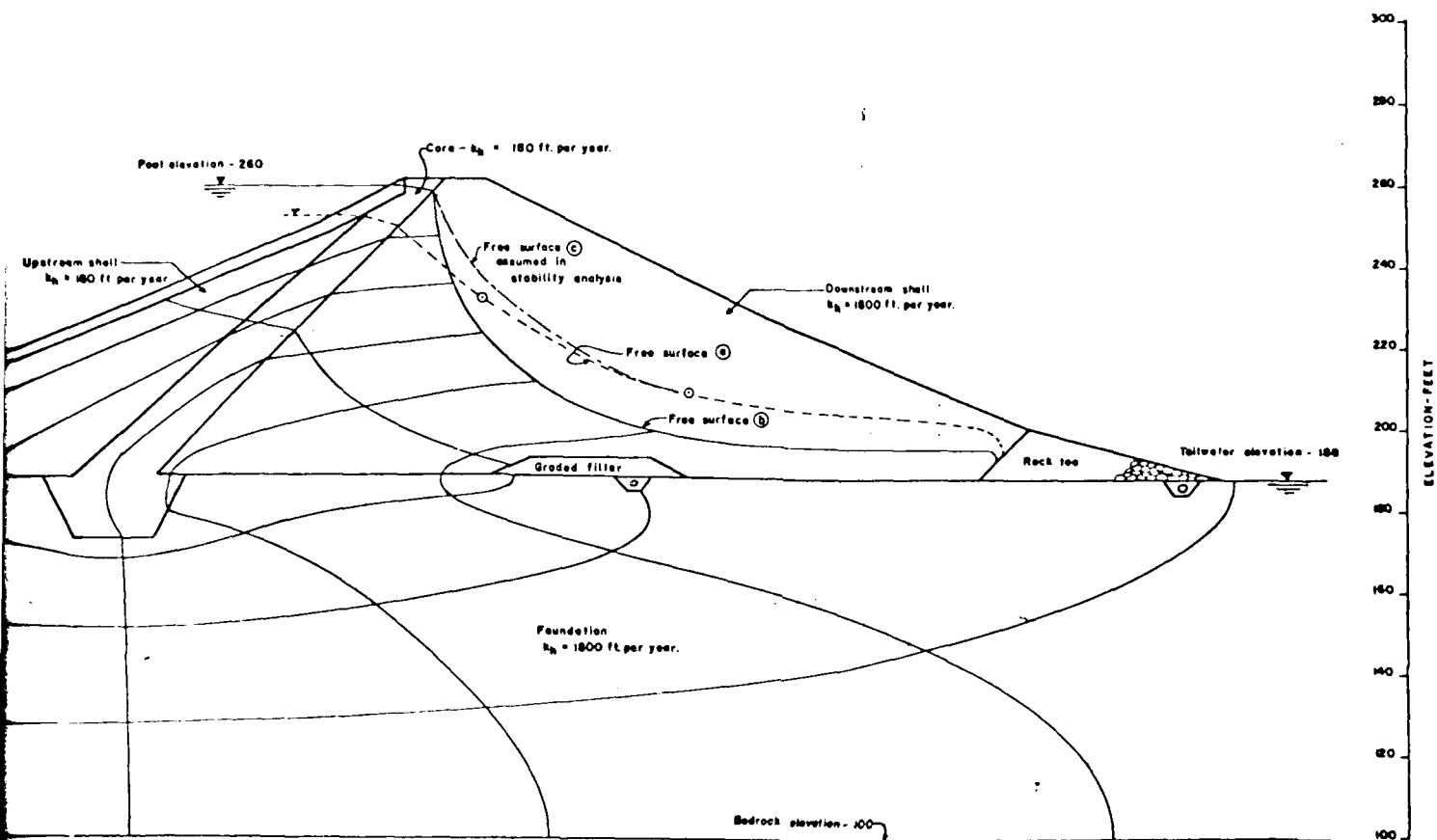
$$Q = \frac{7.5}{365} \times L_e \times \frac{n_f}{n_d} \times H \times \sqrt{k_h k_v}$$

SAMPLE SEEPAGE COMPUTATIONS

- a) Embankment (pool elevation - 260)
 $k_h, k_v = 1800$ ft/yr. $Q_e = 610 \times \frac{180}{6} \times 72 \times \sqrt{1800 \times 270} = \frac{7.5}{365}$
 $H = 72$ ft.
 $n_f = 1.63$ $Q_e = 149,000$ gal per day (plotted above)
 $n_d = 6$
 $L_e = 610$ ft.
- b) Foundation (pool elevation - 260)
 $k_h, k_v = 1800$ ft/yr.
 $H = 72$ ft. $Q_f = 262 \times \frac{180}{6} \times 72 \times \sqrt{1800 \times 200} = \frac{7.5}{365}$
 $n_f = 3.35$ $Q_f = 150,000$ gal per day (plotted above)
 $n_d = 6$
 $L_f = 262$ ft.
- $Q = Q_e + Q_f = 279,000$ gal per day (plotted above)

NOTES

1. \odot These measured values of k_h and k_v are based on flow tests.
2. To obtain the total seepage loss from the flow net, the seepage loss from the flow net is multiplied by the number of flow paths.



FLOW NET AT CROSS SECTION 5+0

NOTES

1. (C) These points shown above are at locations of standpipes where field measurements of free surface were made. Points shown are the maximum values from Figure 3 with a pool elevation of 253.

Dotted line (C) indicates probable location of actual free surface as opposed to solid line (B) which is location of free surface obtained by flow net sketching using the noted permeability assumptions.

2. To obtain free surface (C) and seepage forces that reflect field data for use in stability analysis, the following procedures were used:
 - a) Raise elevation of free surface (C) by approximately 7 feet at the pool and to conform with pool elevation of 260

- b) Start equipotentials at the new surface from the elevation at which they intersect line (B) and extend them to the left (upstream) parallel to their original direction

Seepage forces from the altered flow net are greater than those from the original net. However, only the shallow trial circles in the embankment are materially affected by these increased forces (See location of trial circles in Figure 3)

MIANUS RIVER DAM
STAMFORD, CONNECTICUT

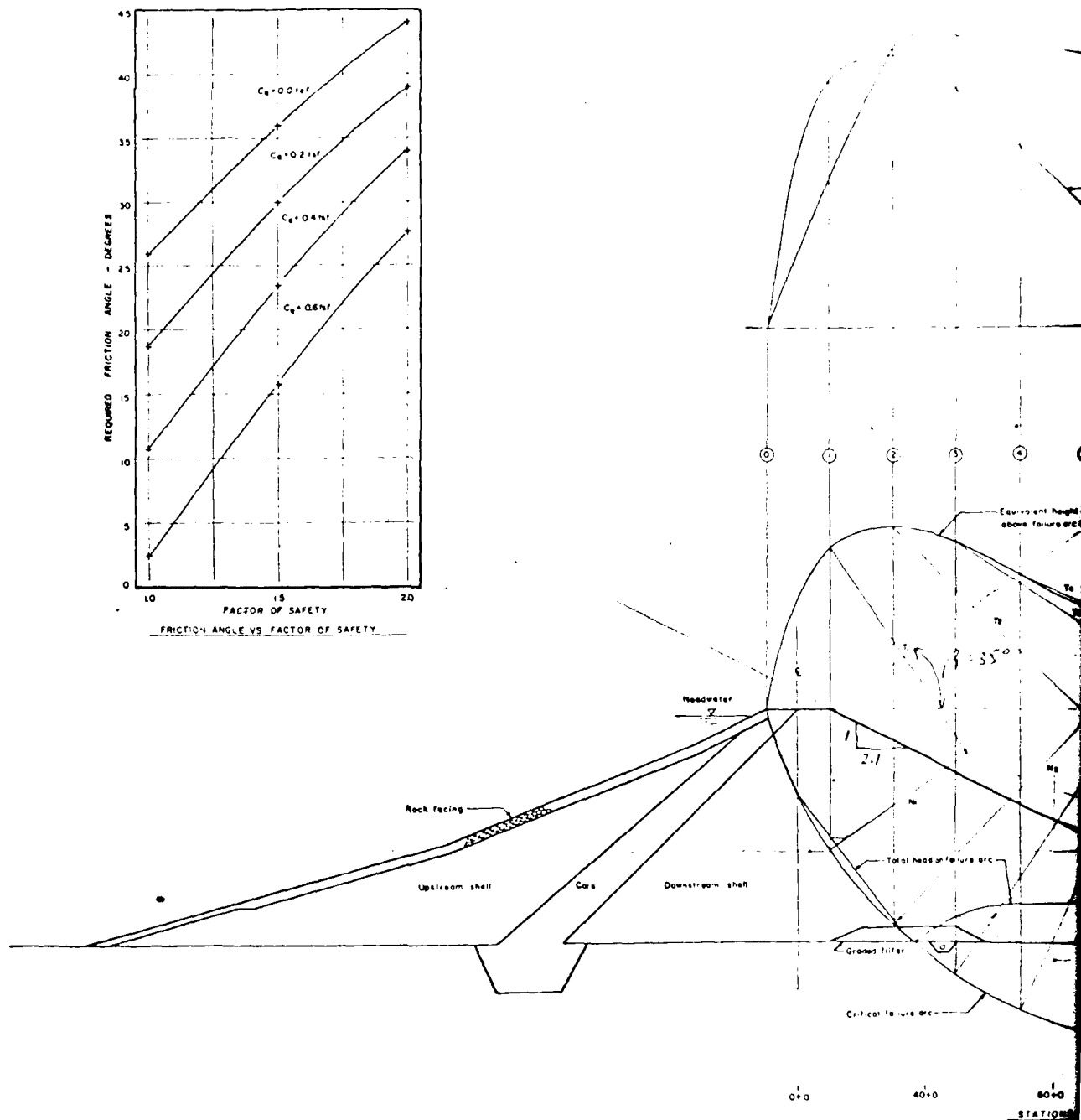
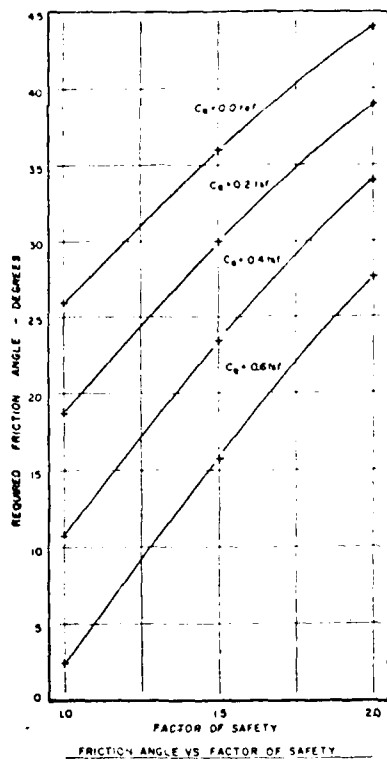
FLOW NET STUDY

STATE OF CONNECTICUT
WATER RESOURCES COMMISSION
STATE OFFICE BUILDING HARTFORD, CONN.

Scale: - 1" = 20'

FEBRUARY

959

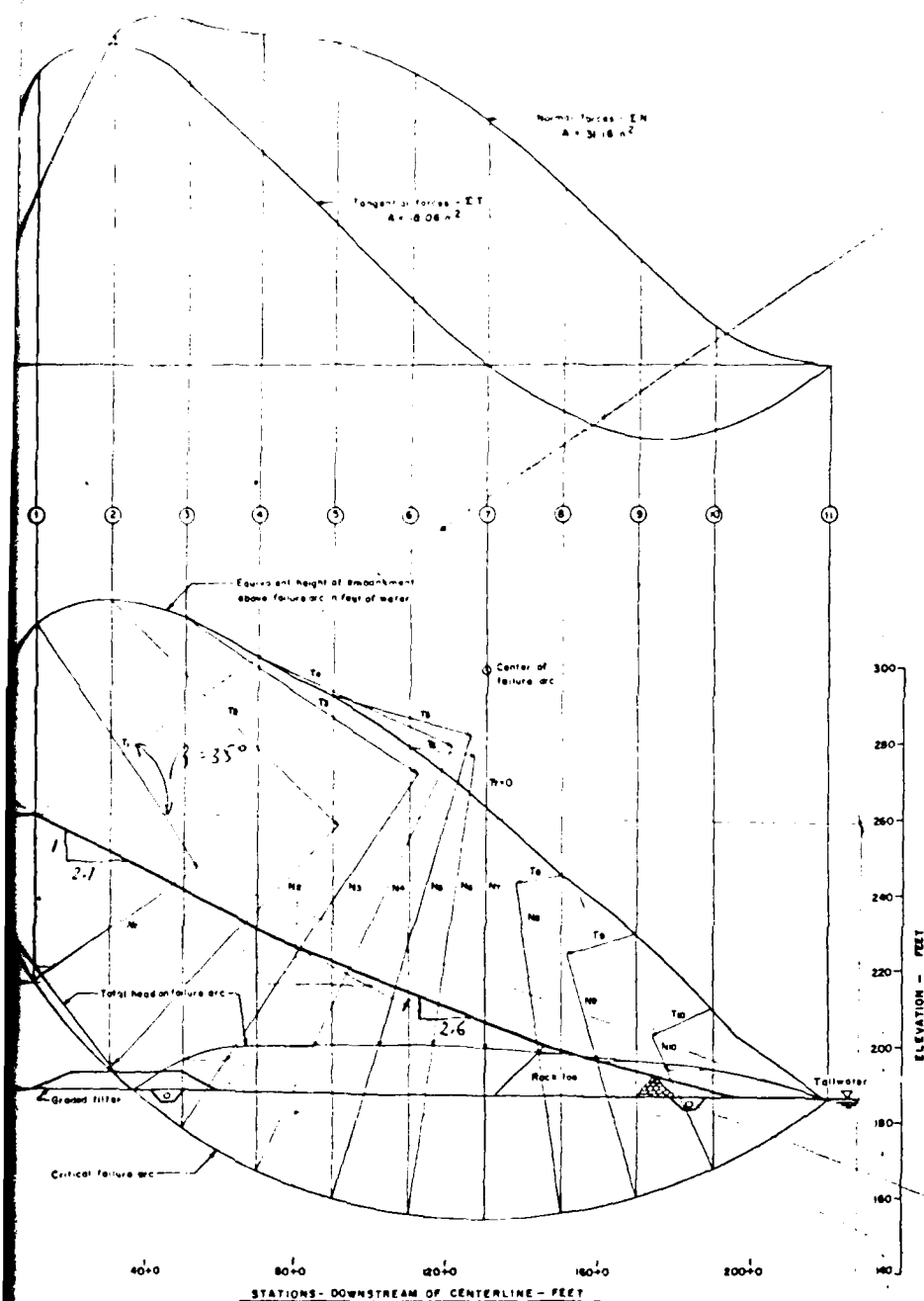


CROSS SECTION AT STA 5+0

Scale 1" = 20' (approx 400 sq ft)

58-211

HALEY & ALLEN
CONSULTING ENGINEERS



ASSUMPTIONS

Embankment	135 lbs per cu foot
Foundation	128 lbs per cu foot
Cohesion	Constant along failure arc
Friction angle	Constant along failure arc
Flow net	See Figure 1

PROPERTIES OF CRITICAL CIRCLE

Radius	145 feet
Central angle	113°
Arc length	286 feet
Location of center	Sta 150+79, Elev 300

COMPUTATION OF FACTOR OF SAFETY

$$F_s = \frac{EN \tan \theta_r + C L_f}{ET} \quad \text{(Factor of safety with respect to strength)}$$

ET = Sum of tangential driving forces.
 EN = Sum of normal interslice forces.
 θ_r = Required effective friction angle.
 C_r = Required effective cohesion.
 L_f = Length of failure arc.

$$ET = 15.06 \times \frac{62.4}{2000} \times 400 = 187.7 \text{ tons}$$

$$EN = 31.16 \times \frac{62.4}{2000} \times 400 = 387.7 \text{ tons}$$

F_s	C_r	$\tan \theta_r$	θ_r
1.0	0.0	0.485	25.8
	0.2	0.337	18.6
	0.4	0.189	10.7
	0.6	0.0414	2.4
1.5	0.0	0.726	36.0
	0.2	0.580	30.1
	0.4	0.432	23.4
	0.6	0.284	15.8
2.0	0.0	0.969	44.0
	0.2	0.823	39.4
	0.4	0.674	34.0
	0.6	0.524	27.7

NOTES

- See Figure 6 for tabulated data on all trial failure circles and their location in the embankment.
- See Figure 1 for flow net used in obtaining total head on failure arc.

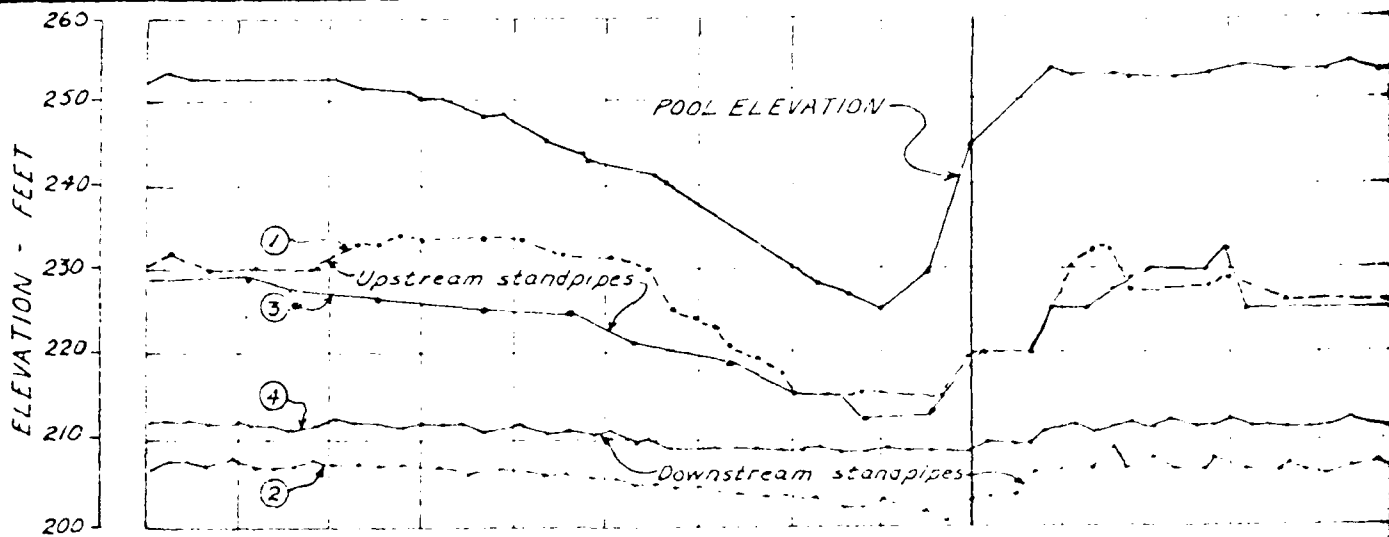
MIANUS RIVER DAM STAMFORD, CONNECTICUT STABILITY ANALYSIS

STATE OF CONNECTICUT
 WATER RESOURCES COMMISSION
 STATE OFFICE BUILDING HARTFORD, CONNECTICUT
 SCALE AS SHOWN

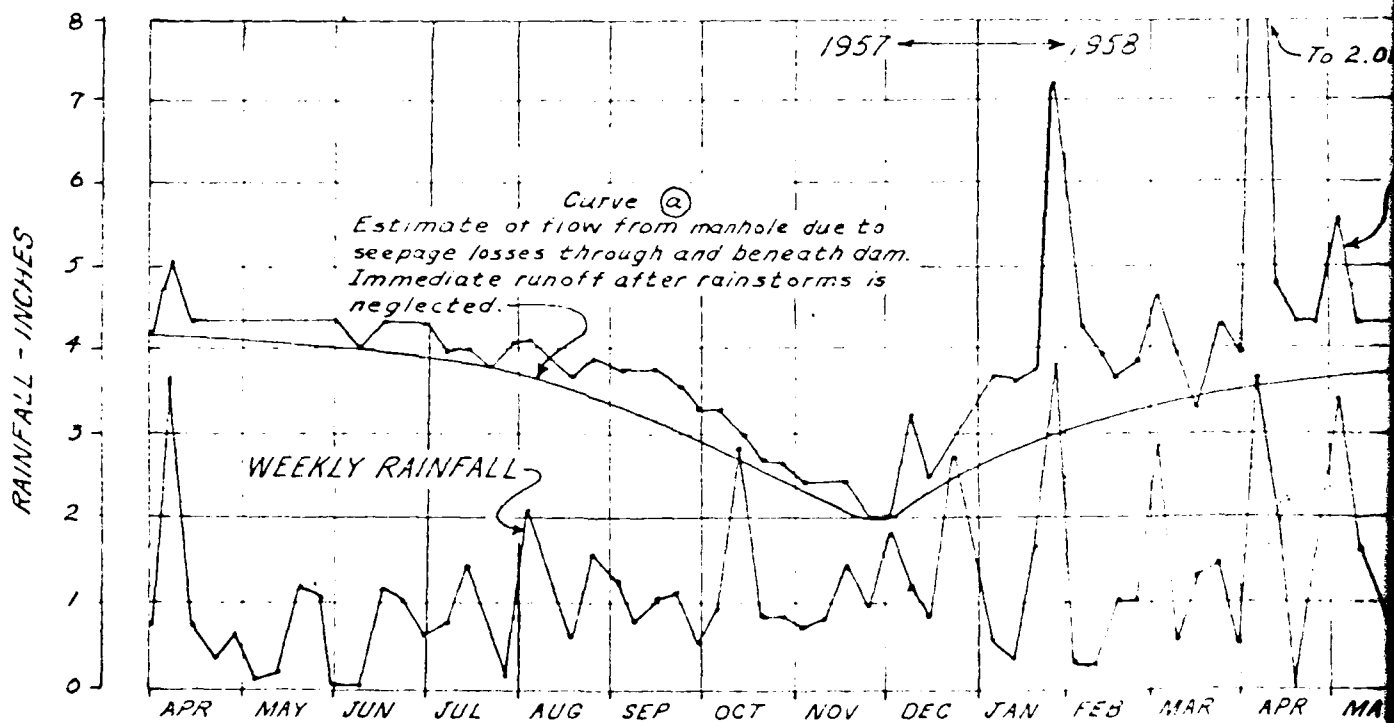
FEBRUARY

1959

FIGURE 2



OBSERVATIONS OF FREE WATER SURFACE IN EMIBANI

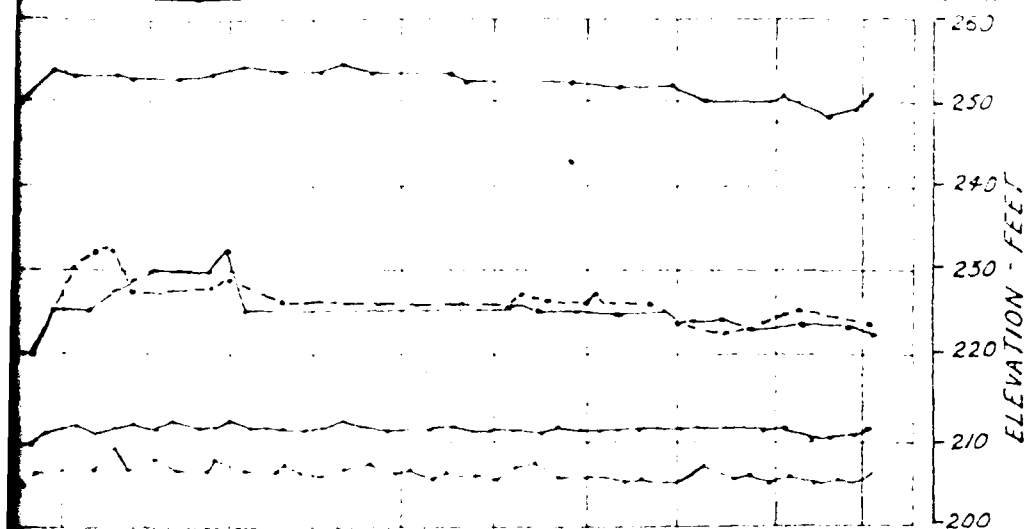


1957 ← | → 1958

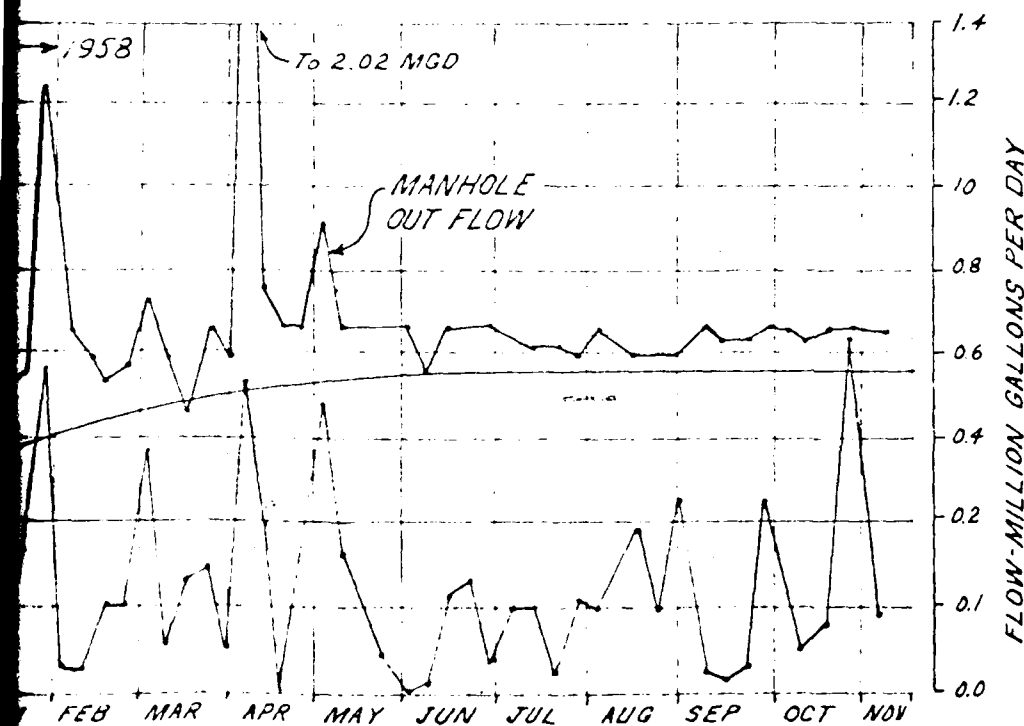
RAINFALL AND MANHOLE OUTFLOW

58-211

HALEY & ALDRICH
CONSULTING ENGINEERS



SURFACE IN EMBANKMENT



1958
MANHOLE OUT FLOW

NOTES:

1. Data plotted hereon is obtained from field data sheets made available by the Water Resources Commission of Connecticut. Data was taken by the Greenwich Water Company

2 See Figure 4 for location of standpipes.

MIANUS RIVER DAM
STAMFORD, CONNECTICUT
**PLOT OF PREVIOUS
FIELD MEASUREMENTS**

FEBRUARY

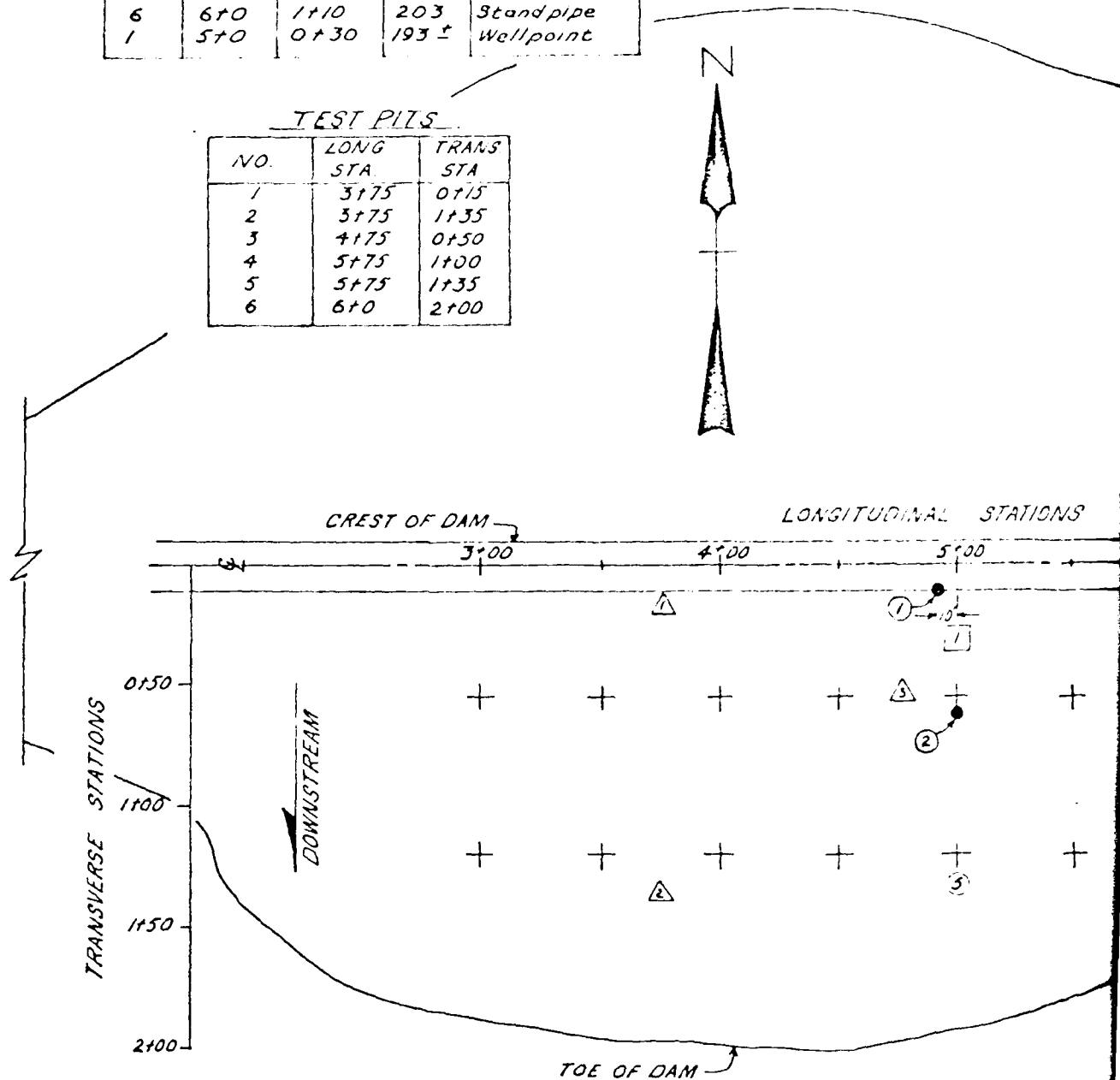
1959

WATER OBSERVATIONS

NO.	LONG. STA.	TRANS. STA.	BOT. ELEV.	REMARKS
5	5+0	1+30	195	Standpipe
6	6+0	1+10	203	Standpipe
1	5+0	0+30	193 ±	Wellpoint

TEST PILLS

NO.	LONG. STA.	TRANS. STA.
1	3+75	0+15
2	3+75	1+35
3	4+75	0+50
4	5+75	1+00
5	5+75	1+35
6	6+0	2+00



PLAN VIEW OF DAM

Scale: 1" = 60'

58-211

HALEY & A. DRICH
CONSULTING ENGINEERS

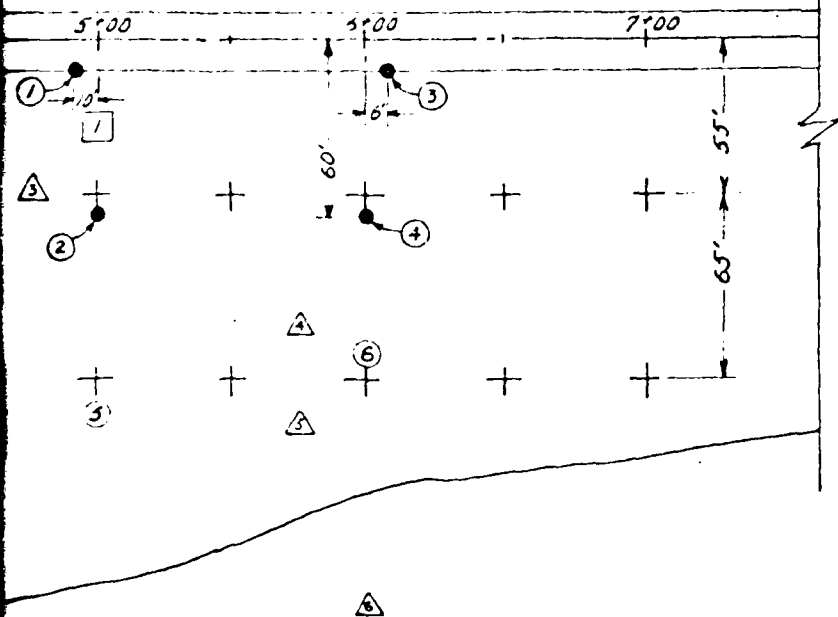
NOTES:

1. See Figure 5 for details of wellpoint and displacement stakes.
2. Wellpoint is to be installed in graded filter.
3. See text for description of test pits
4. Test pit No 5 is to be driven into rock toe so that rock is exposed.
5. Test pit No 6 may be relocated as necessary to obtain representative samples of the foundation material.
6. Displacement stakes in each row are spaced 50' on center.

LEGEND:

- Wellpoint
- △ Test Pits
- 's New Standpipes
- + Displacement stakes
- Existing Standpipes

LONGITUDINAL STATIONS



PLAN VIEW OF DAM

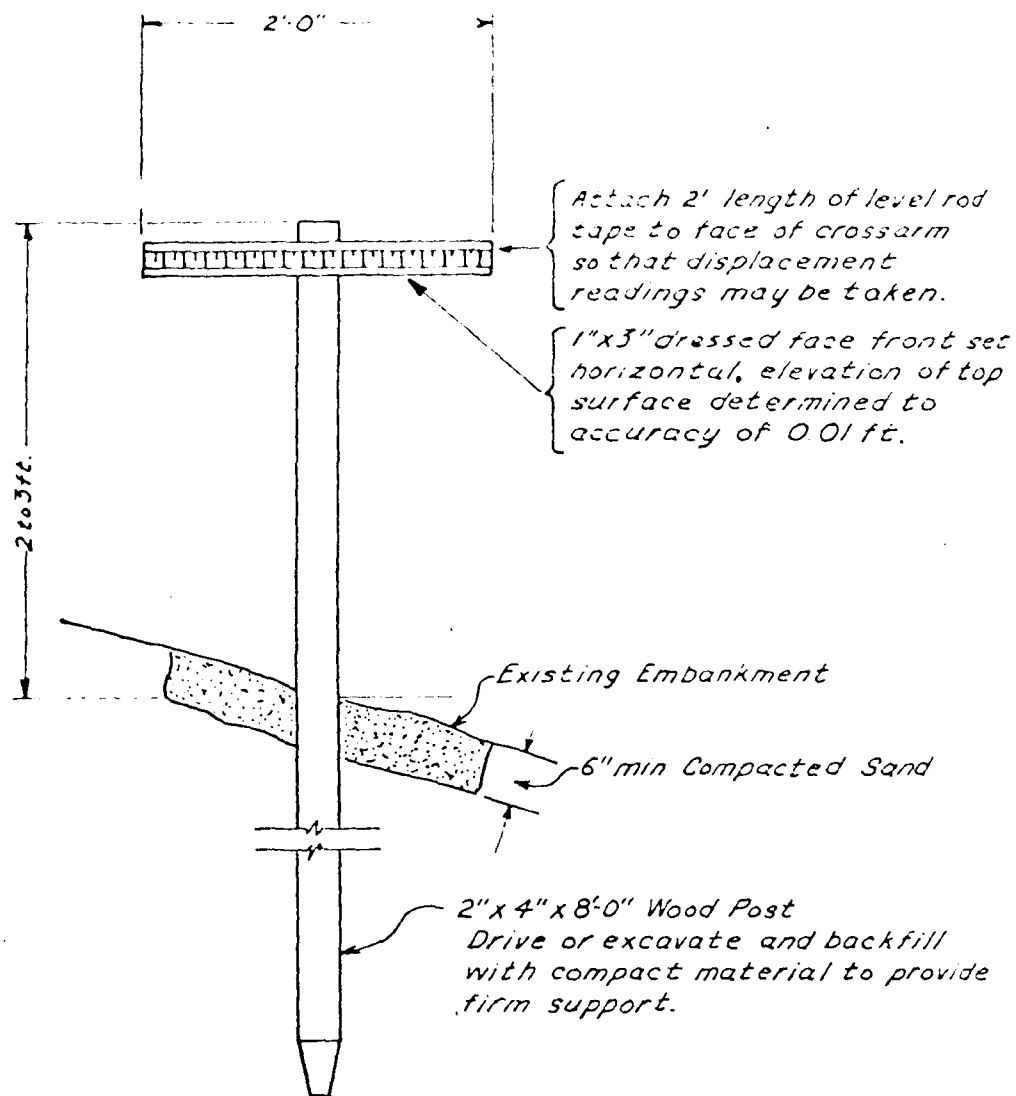
1" = 60'

MIANUS RIVER DAM
STAMFORD, CONN.

LOCATION PLAN
FIELD INVESTIGATIONS

FEBRUARY

1959

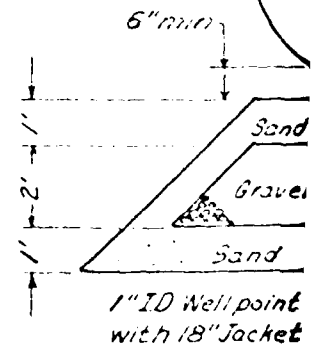


Downstream slope of Dam

1" Riser

Backfill with material similar to embankment and tamp in layers as casing is removed

Detail "A"



TYPICAL DISPLACEMENT STAKE

(Not to Scale)

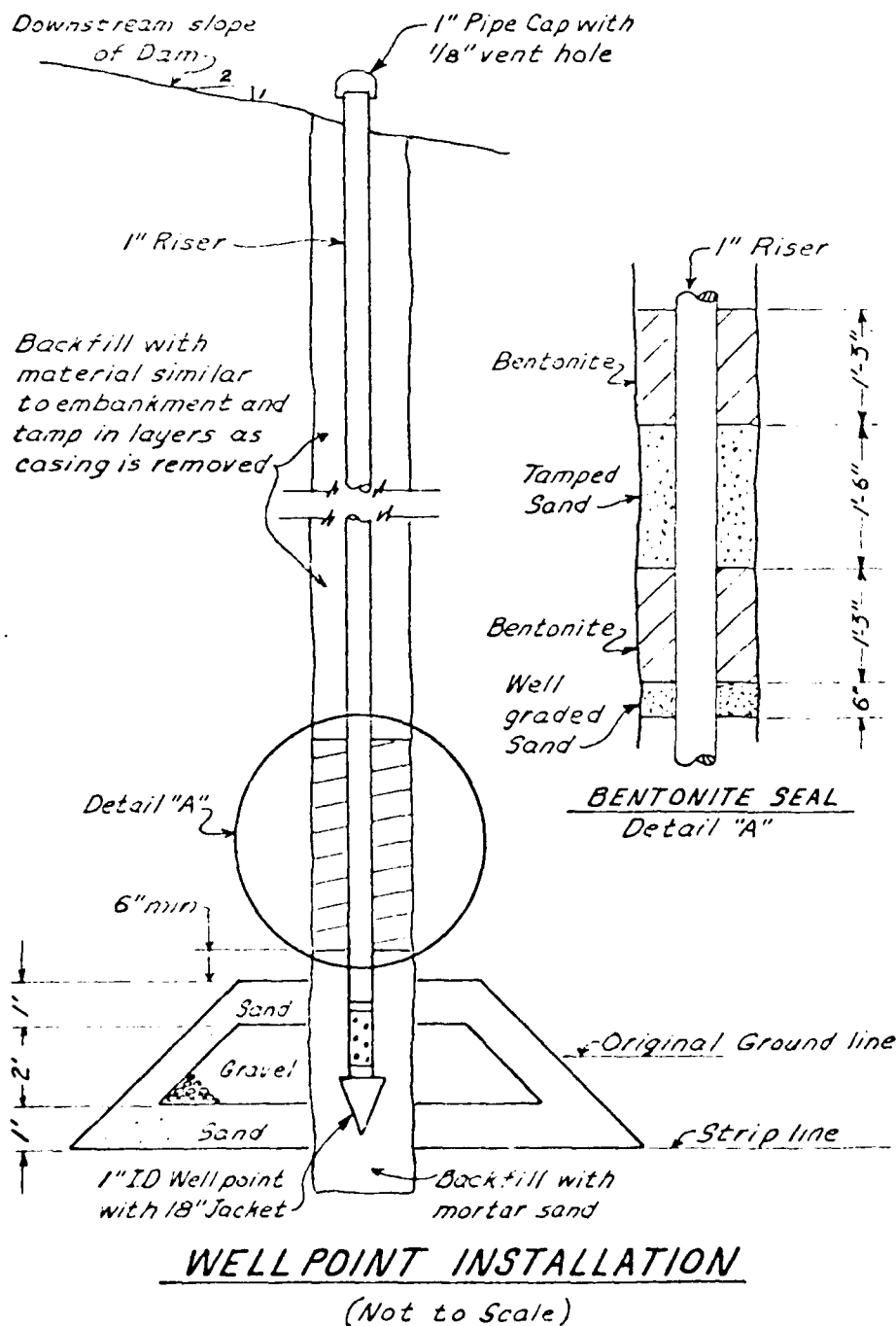
WELL

58-211

HALEY & ALDRICH

NOTES:

1. Bentonite compacted in 3" layers with 3/4" layer of pea stone on top of each layer to prevent Bentonite from sticking to tamping head. Bentonite formed in 3/8" balls near plastic limit and dropped into drive casing. Bentonite seal placed as casing is withdrawn.
2. Drive wellpoint into graded filter so that gravel surrounds jacket.



MIANUS RIVER DAM
STAMFORD, CONN.

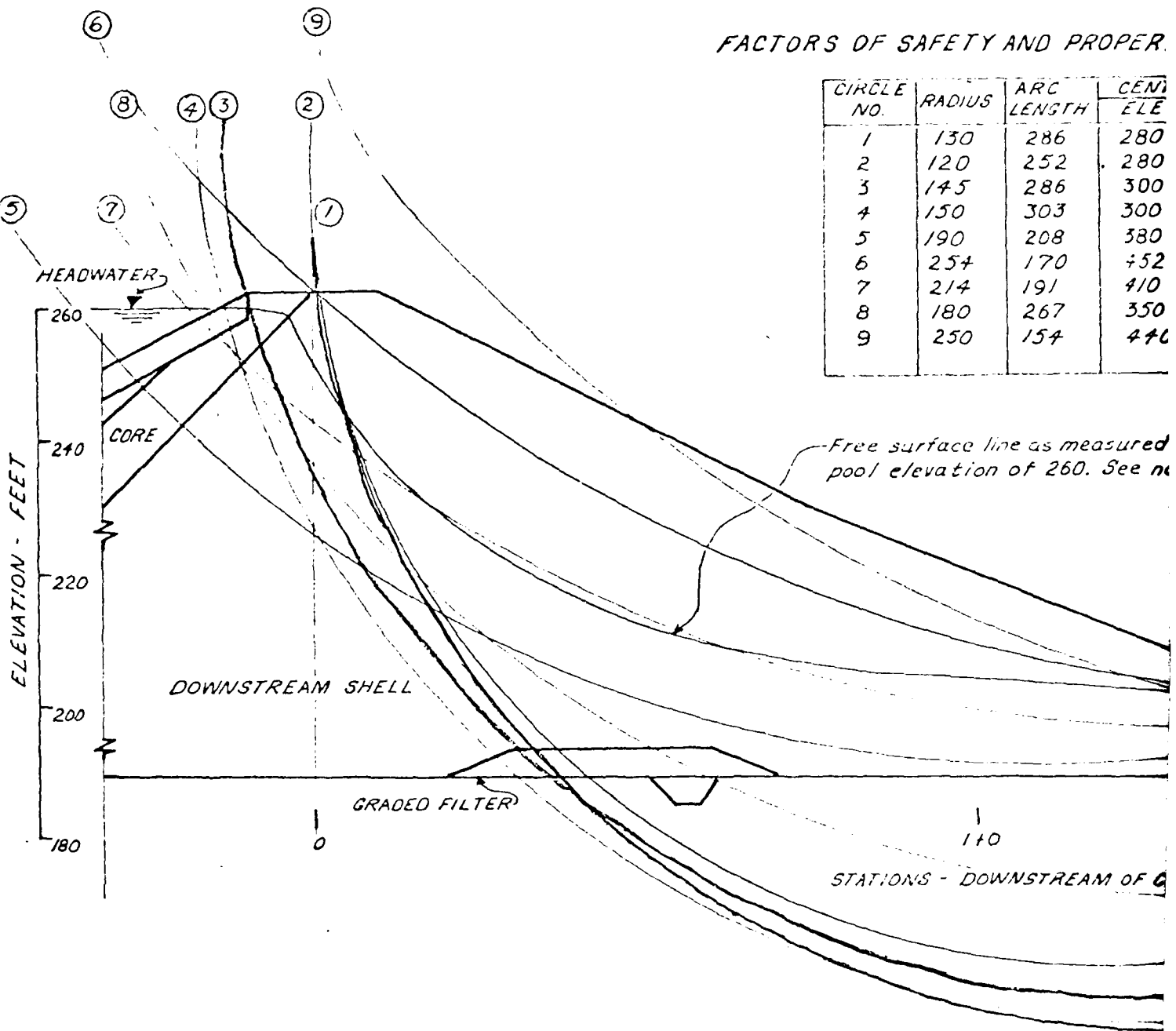
DISPLACEMENT STAKES AND WELLPOINT INSTALLATION

FEBRUARY

1959

FACTORS OF SAFETY AND PROPER.

CIRCLE NO.	RADIUS	ARC LENGTH	CEN. ELE
1	130	286	280
2	120	252	280
3	145	286	300
4	150	303	300
5	190	208	380
6	254	170	452
7	214	191	410
8	180	267	350
9	250	154	446



TRIAL CIRCLES CROSS SECTION AT STA. 5+0

SCALE: 1" = 20'

58-211

HALEY & ALDRICH

CHARTERED SURVEYORS

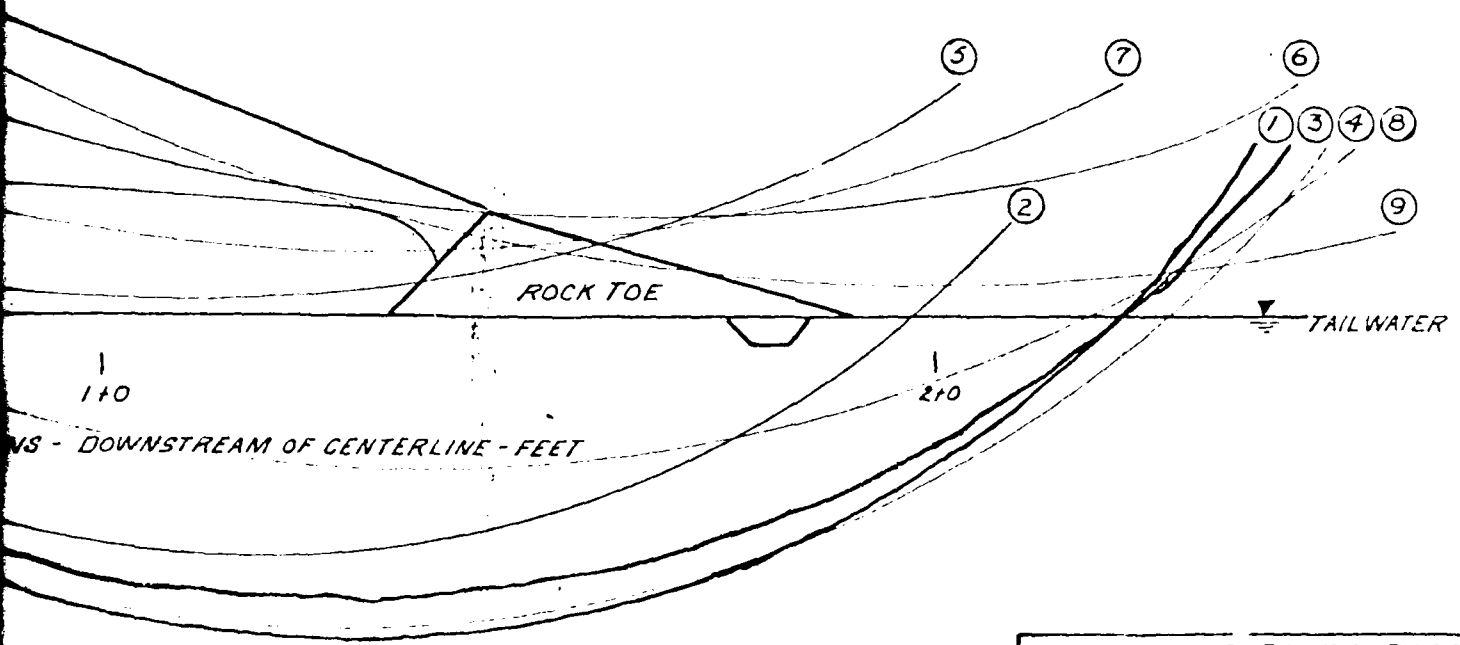
SAFETY AND PROPERTIES OF TRIAL CIRCLES

RADIUS	ARC LENGTH	CENTER LOCATION		FACTOR OF SAFETY
		ELEV.	STA.	
130	286	280	1+30	1.45
120	252	280.	1+20	1.57
145	286	300	1+30	1.45.
150	303	300	1+30	1.49.
190	208	380	1+10	1.94
254	170	452	1+66	1.61
214	191	410	1+34	1.68
180	267	350	1+40	1.55
250	154	440	2+00	1.82

NOTES:

- 1. See Figure 2 for stability analysis of the critical circle, No 3.
- 2. See Figure 1 for flow net used in all trials
- 3. Factors of safety are computed for case of zero cohesion and friction angle of 35°
- 4. Stations for locating centers of trial circle are measured downstream of dam centerline.

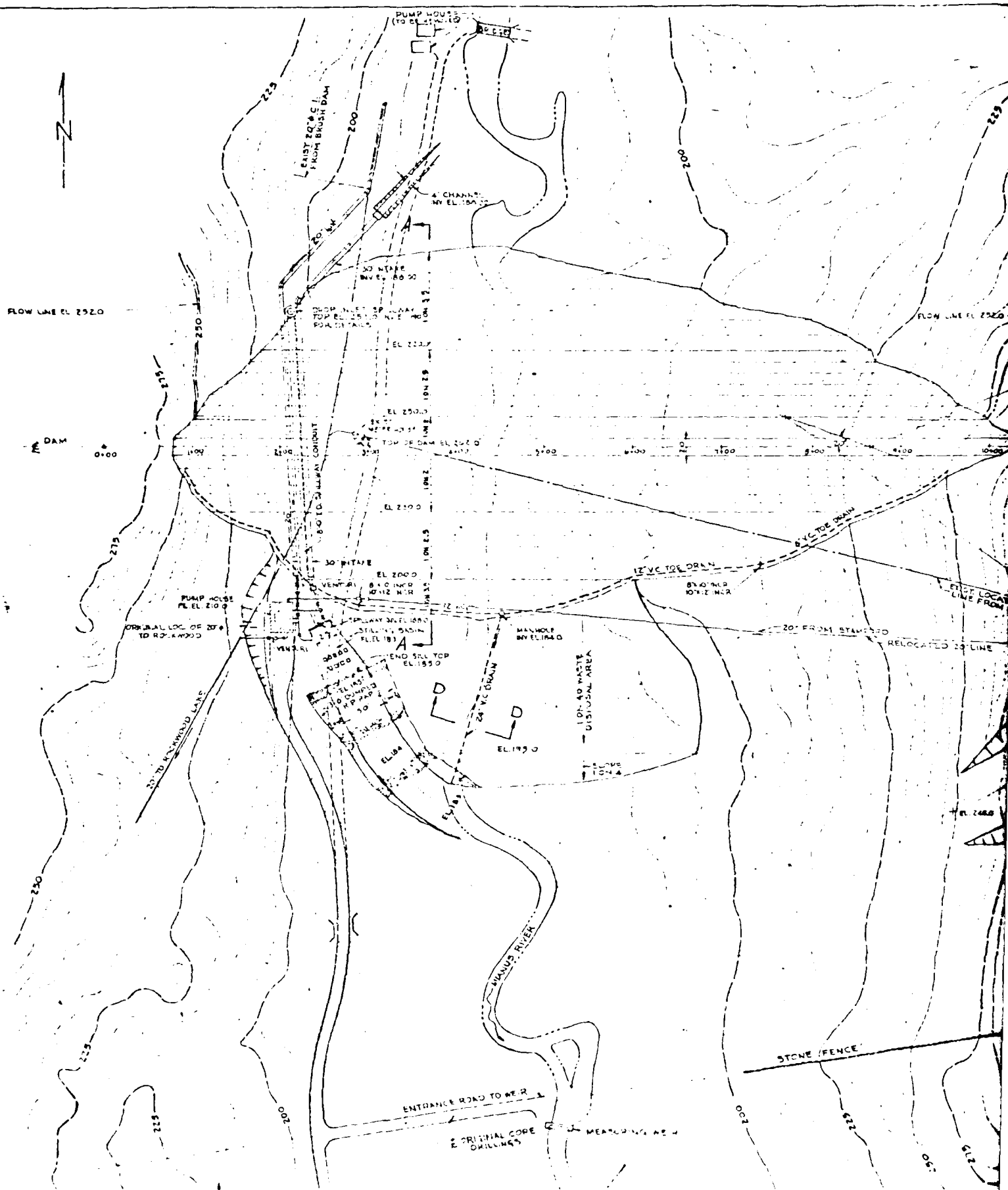
surface line as measured and modified for elevation of 260. See notes of Figure 1.

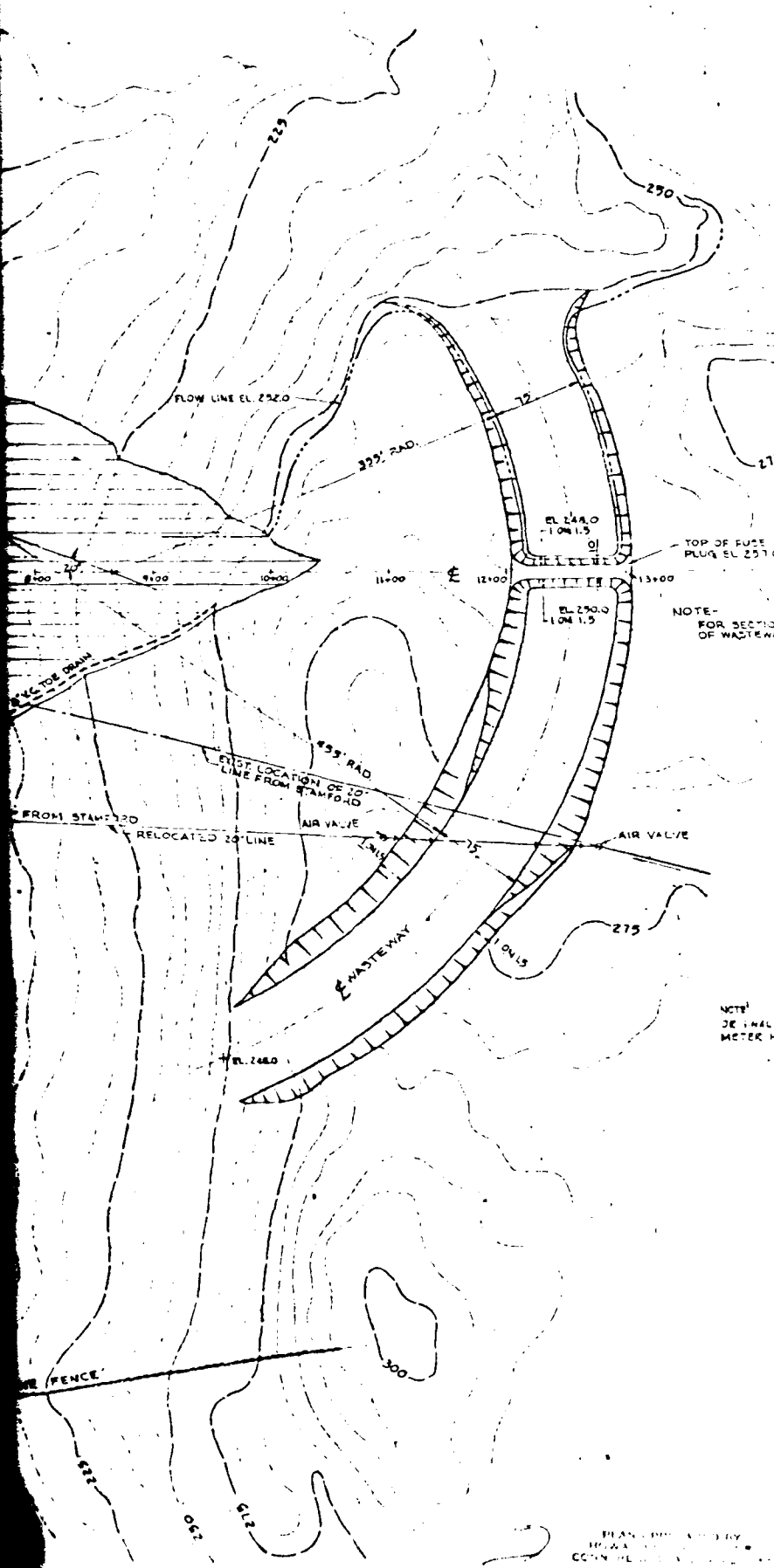


TRIAL CIRCLES
ON AT STA. 5+0
1" = 20'

MIANUS RIVER DAM
STAMFORD, CONNECTICUT
TRIAL CIRCLES
FOR
STABILITY ANALYSIS
FEBRUARY 1959

FIGURE 6



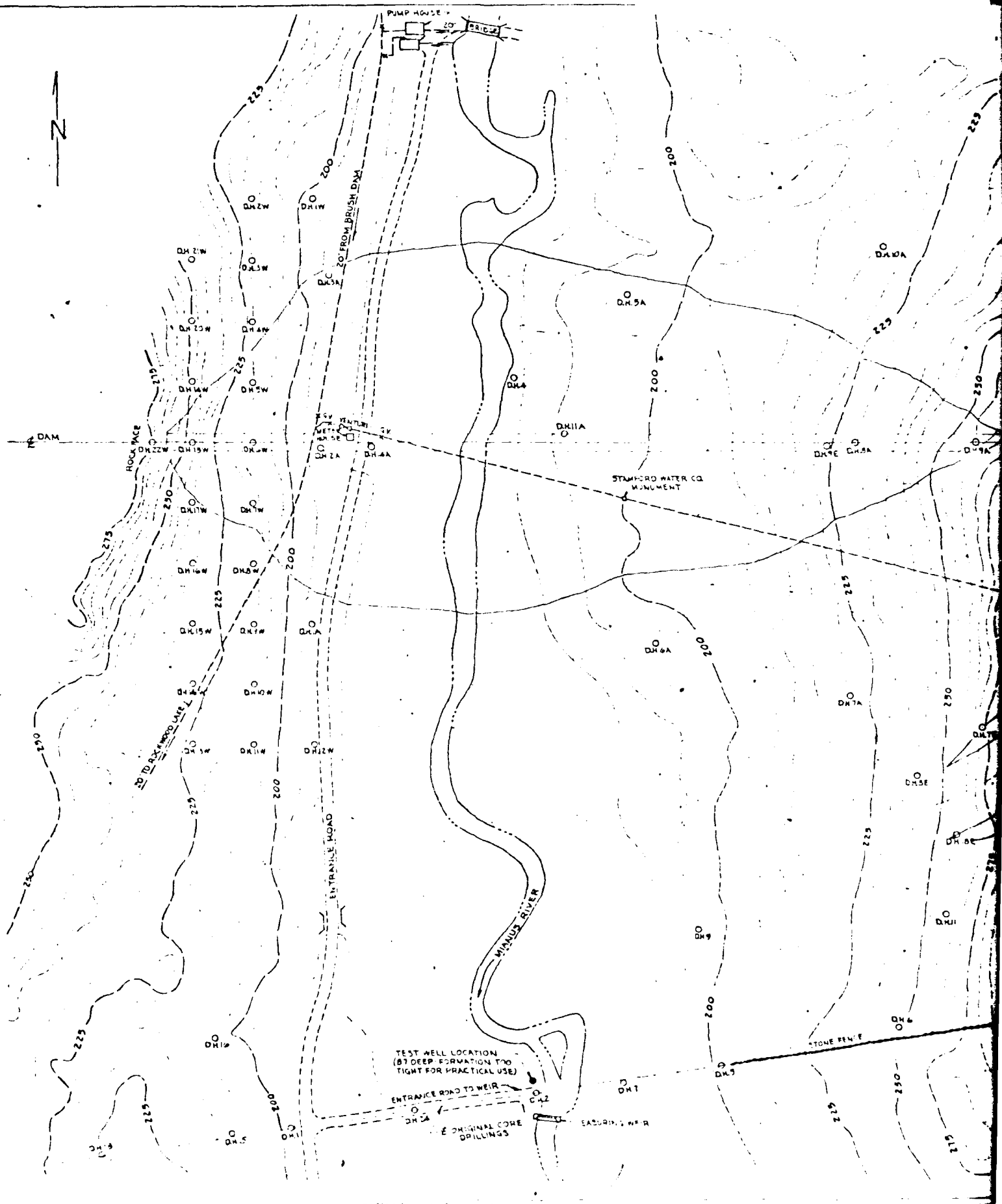


NOTE:
FOR SECTIONS AND PROFILE
OF WASTEWAY SEE DWG. No. 245-9

NOTE:
DE SIGN & STAKE AT EXISTING
METER HOUSE STA 3+00

REVISIONS	MIANUS RIVER DAM	
	GENERAL PLAN	
	GREENWICH WATER COMPANY GREENWICH, CONN.	
	AMERICAN WATER WORKS SERVICE COMPANY INC. 181 SOUTH BRIDGE STREET PHILADELPHIA, PA.	
	SCALE: 1" = 40' HORIZ. 1" = 10' VERT.	DATE: 12-1-55 DRAWN BY: J.E.T. CHECKED BY: J.E.T. APPROVED BY: J.E.T.

PLAN AND SECTION BY
HOWARD A. JONES
CONN. REG. NO. 10000



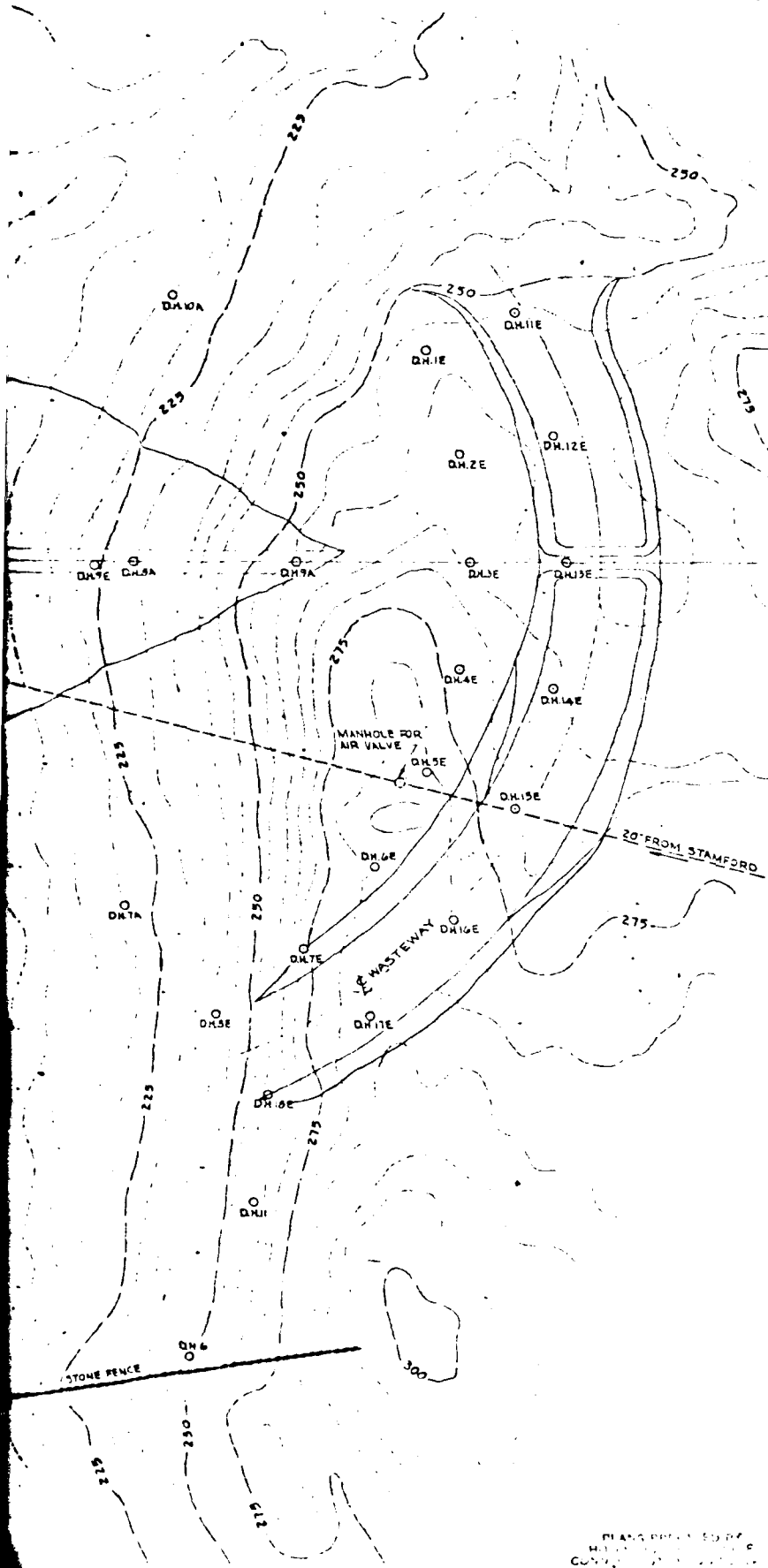
TEST HOLE DATA

NO	GROUND ELEV.	DEPTH	ROCK ELEV.
1A	192.25	28	164.25
2A	192.12	23	169.12
3A	193.50	107	DRILLED
4A	189.12	30	159.12
5A	197.76	46	151.76
6A	198.08	45	153.08
7A	220.42	18	202.42
8A	229.53	20	209.50
9A	257.10	15	242.10
10A	216.31	16	200.31
11A	196.11	31	145.11
4	190.8	73	117.8

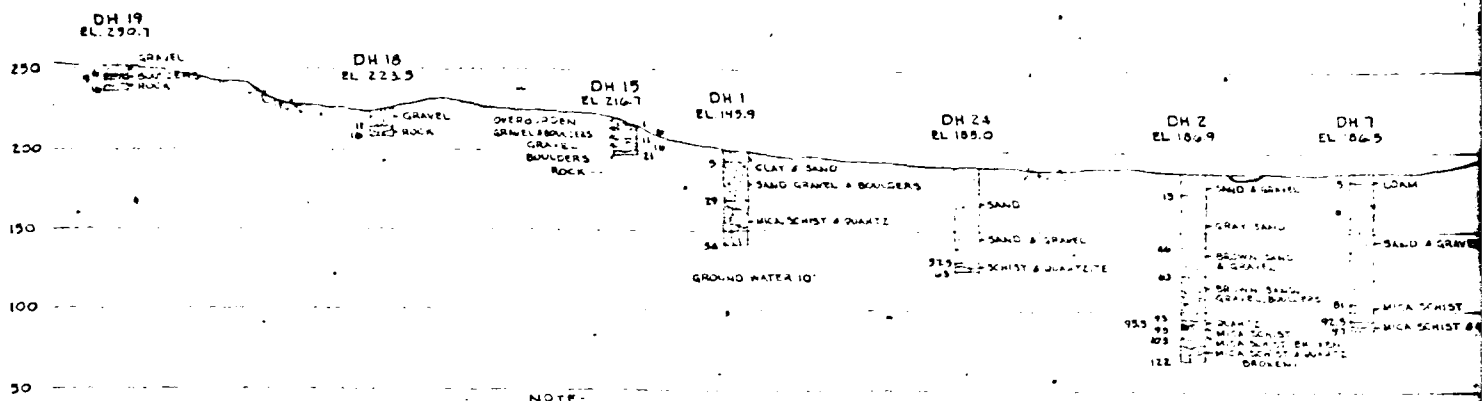
NO	GROUND ELEV.	DEPTH	ROCK ELEV.
1	199.9	29	169.9
2	186.9	93	93.9
3	202.1	53	149.1
6	250.9	4	246.9
7	186.5	61	125.5
9	197.7	13	184.7
11	263.9	10	253.9
15	216.7	16	200.7
16	209.7	11	198.7
18	223.5	11	212.5
24	188.0	57.5	130.5

NO	GROUND ELEV.	DEPTH	ROCK ELEV.
1W	195.60	32	163.6
2W	210.54	13	197.54
3W	216.24	12	204.24
4W	218.69	11	207.69
7W	221.29	14	207.29
8W	212.90	16	196.9
1W	220.39	10	210.34
8W	221.81	15	206.81
9W	221.95	25	196.95
10W	217.95	21	190.95
11W	215.44	30	185.44
12W	193.30	46	147.30
13W	224.68	13	211.68
14W	226.18	9	217.18
15W	227.58	20	207.58
16W	229.66	15	214.66
17W	234.61	21	213.61
18W	244.55	NONE	
19W	247.35	7	240.35
20W	246.90	7	239.90
21W	241.25	13	228.25
22W	273.20	0	ON ROCK

NO	GROUND ELEV.	DEPTH	ROCK ELEV.
1E	257.25	135	243.75
2E	263.10	13	250.10
3E	264.07	14	250.07
4E	272.10	14	258.10
5E	282.20	12	270.20
6E	281.58	16	265.58
7E	273.50	15	258.50
8E	242.34	13	229.34
9E	225.00	16	209.00
11E	235.28	7.5	247.78
12E	258.35	87	247.48
13E	267.40	9	258.40
14E	265.67	14.5	251.17
15E	271.10	17	254.10
16E	279.55	11	268.55
17E	281.53	9	272.53
18E	261.34	9	252.34



REVISIONS	MIANUS RIVER DAM TEST HOLE DATA AND EXISTING FACILITIES	
	GREENWICH WATER COMPANY GREENWICH, CONN.	
	AMERICAN WATER WORKS SERVICE COMPANY, INC. 121 SOUTH BRIDGE STREET, PHILADELPHIA, PA.	
	SCALE: 1" = 40'	USE DIMENSIONS IN FEET
DATE: 7-5-55	CHECKED BY:	
APPROVED BY:	PROJECT NO. 5	INSP. BY:
16 APR 1956		



NOTE:
SEE PLAN BELOW FOR LOCATION
ALSO DRAWING 30 265.5

LOG OF DRILL HOLES ALONG ORIGINAL CENTER LINE SCALE: 1" = 50'

DH 8
EL 192.0

SAND, GRAVEL
 NOT TO ROCK

DH 9
EL 197.7

SAND & GRAVEL
 NOT TO ROCK

DH 10
EL 204.1

BOULDER QUARTZITE,
 SAND & GRAVEL
 QUARTZITE & SCHIST

DH 20
EL 257.2

GRAVEL
 GRAVEL & BOULDERS
 ROCK

DH 21
EL 255.4

GRAVEL
 GRAVEL & BOULDERS
 ROCK

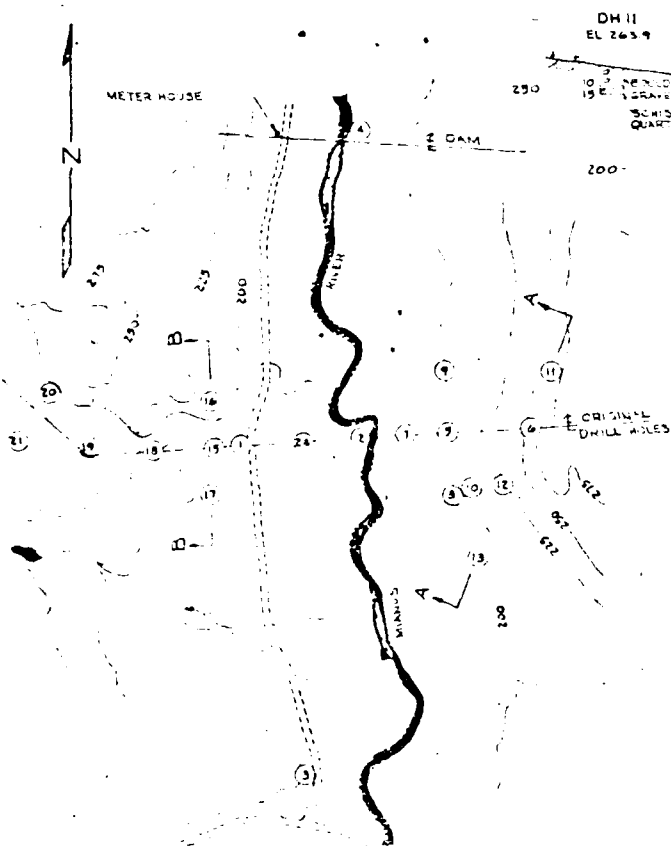
DH 3
EL 185.9

BROWN SAND
 & GRAVEL
 SAND GRAVEL
 & BOULDERS
 QUARTZ (BROKEN)

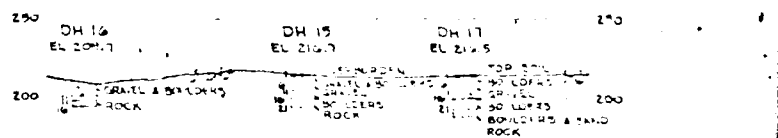
DH 4
EL 190.8

SAND
 GRAVEL
 BOULDERS
 QUARTZ

LOG OF MISCELLANEOUS DRILL HOLES SCALE: 1" = 50'



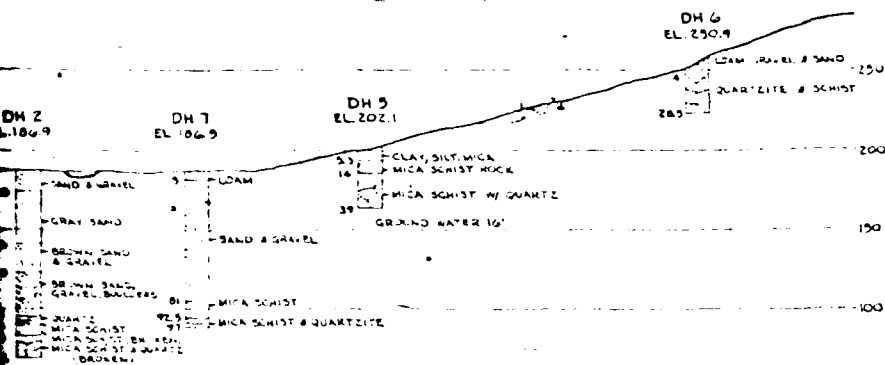
SECTION A-A SCALE: 1" = 50'



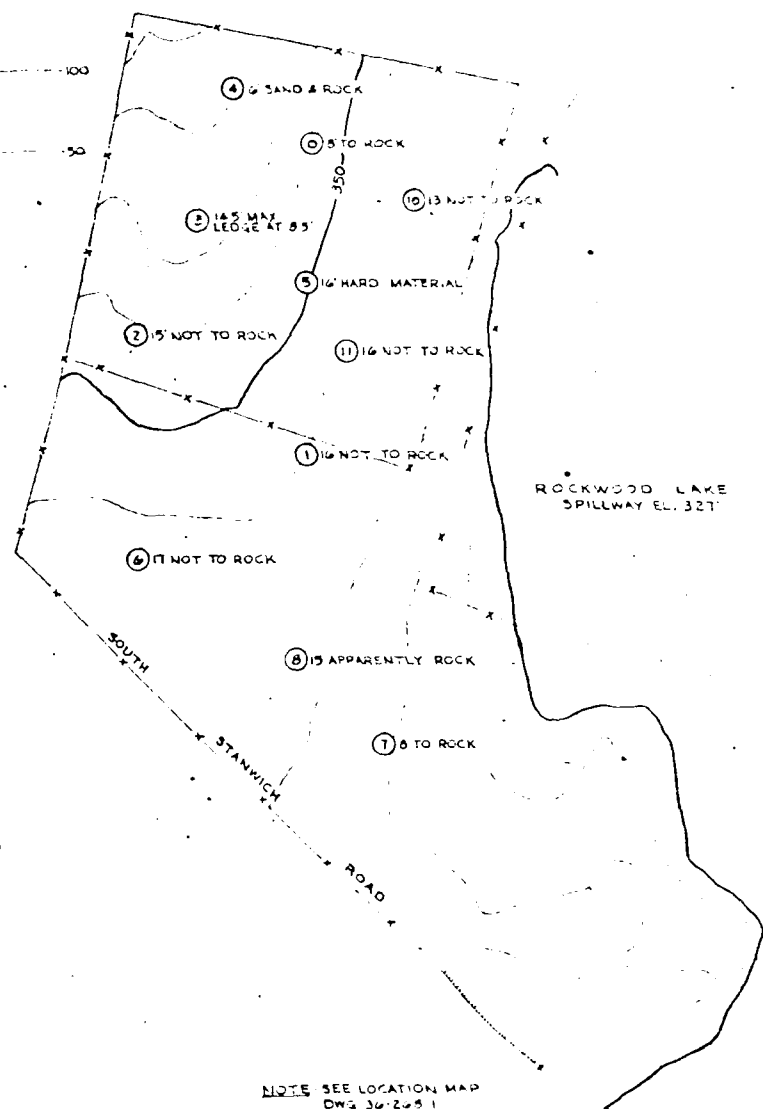
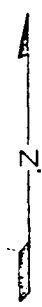
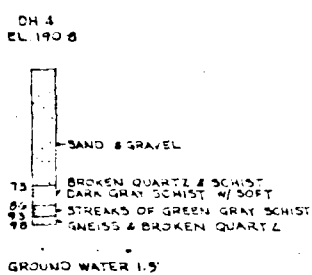
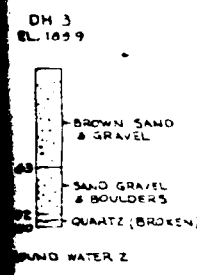
SECTION B-B SCALE: 1" = 50'



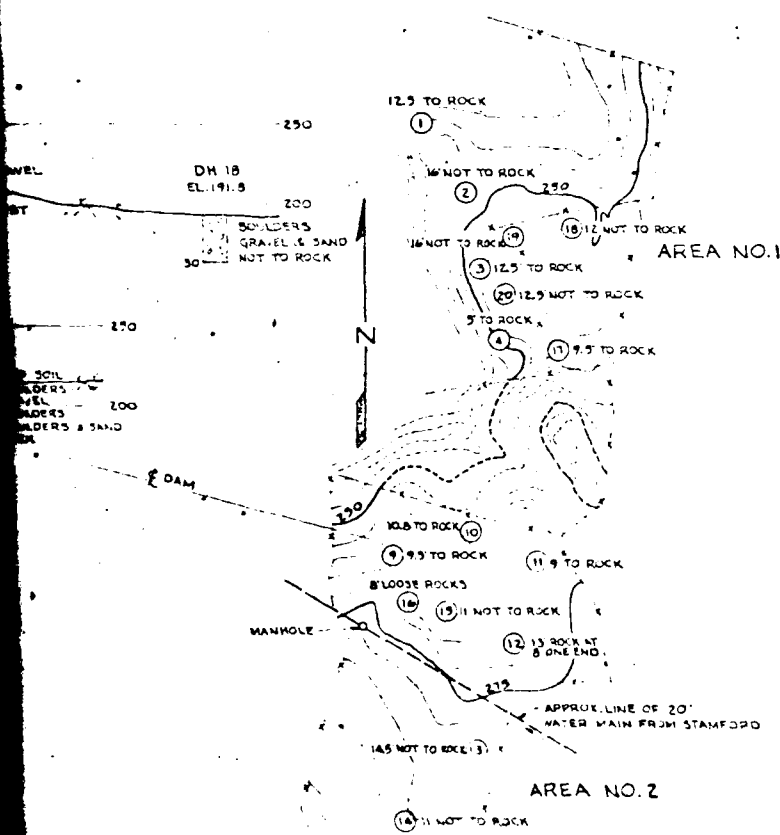
PLAN OF ORIGINAL DRILL HOLES



WILL HOLES
CENTER LINE
F.50

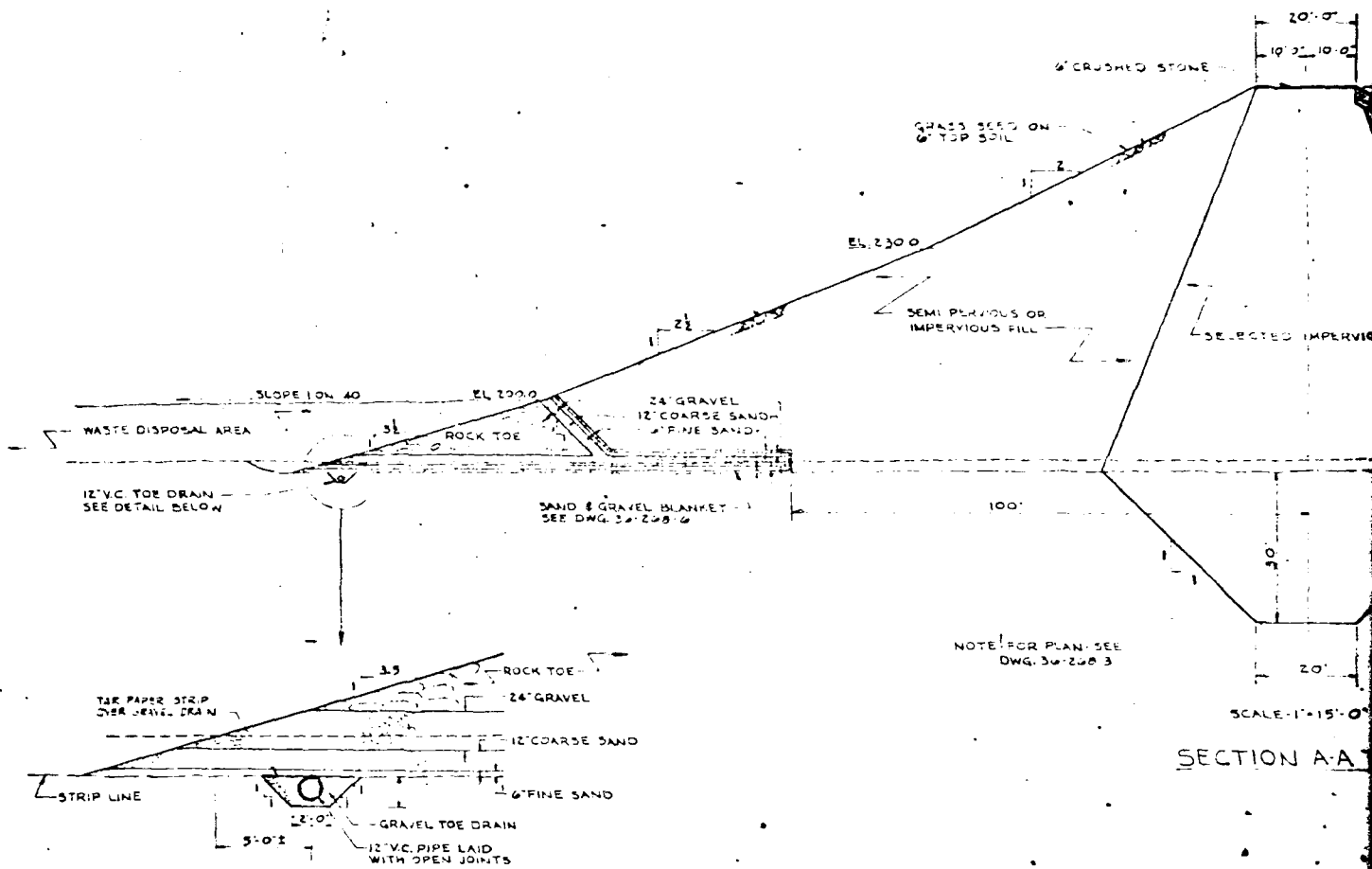


NOTE SEE LOCATION MAP
DWG 36-245.1



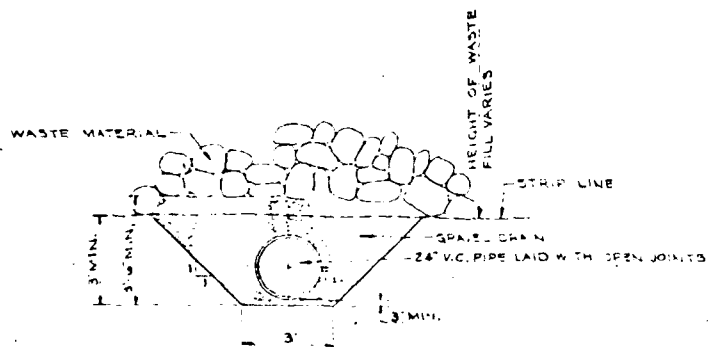
NOTE SEE LOCATION MAP
DWG 36-245.1
PLAN OF TEST PITS
BORROW AREA NO. 3

REVISIONS	MIANUS RIVER DAM PLAN & LOG OF TEST HOLES ON ORIGINAL CENTERLINE PLAN OF BORROW AREA GREENWICH WATER CONTROL GREENWICH, N.Y.
1	AMERICAN WATER WORKS CO. INC. 121 SOUTH 9TH ST. PHILADELPHIA, PA.
2	SCALE: AS SHOWN DRAWN BY: [blank] CHECKED BY: [blank]

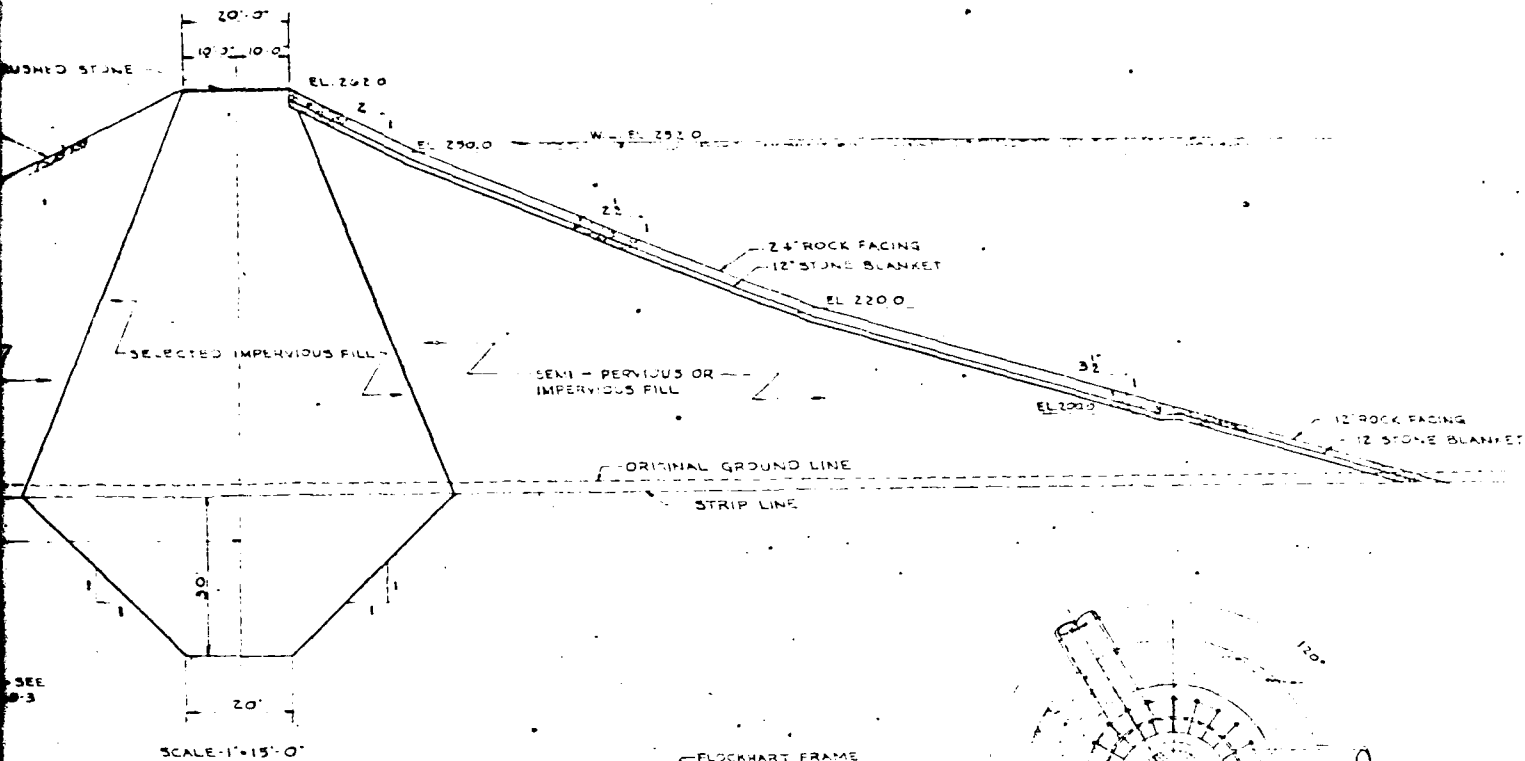


SECTION A-A

DETAIL OF TOE DRAIN
SCALE: 1" = 4'-0"

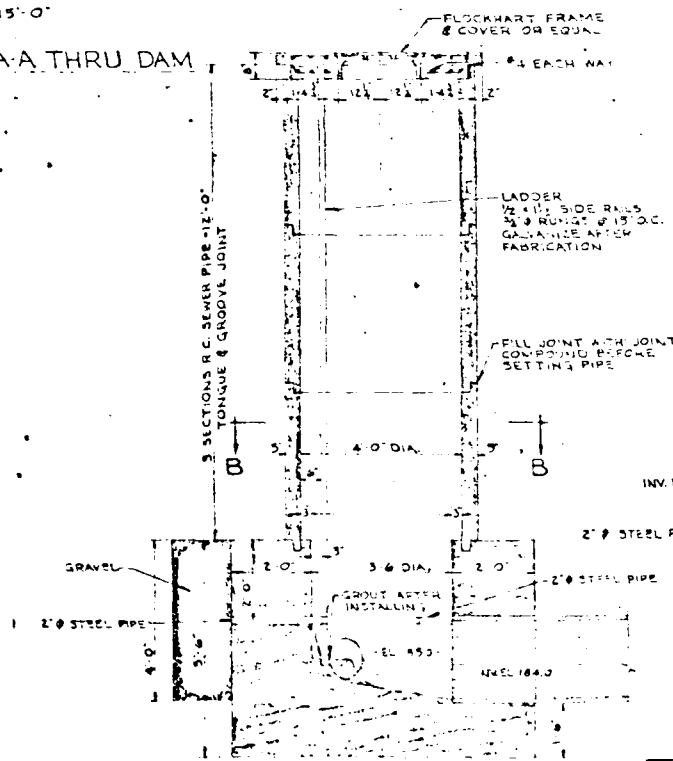


SECTION D-D
SCALE: 1" = 4'-0"



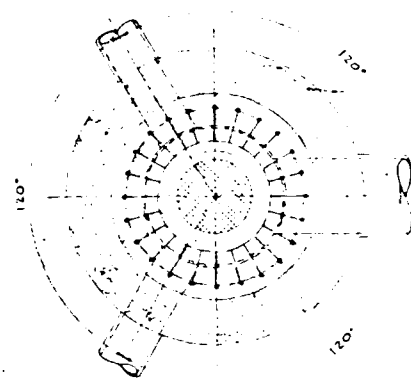
SECTION A-A THRU DAM

SCALE: 1"=15'-0"



SECTION OF MANHOLE

SCALE: 1"=1'-0"



TOP VIEW

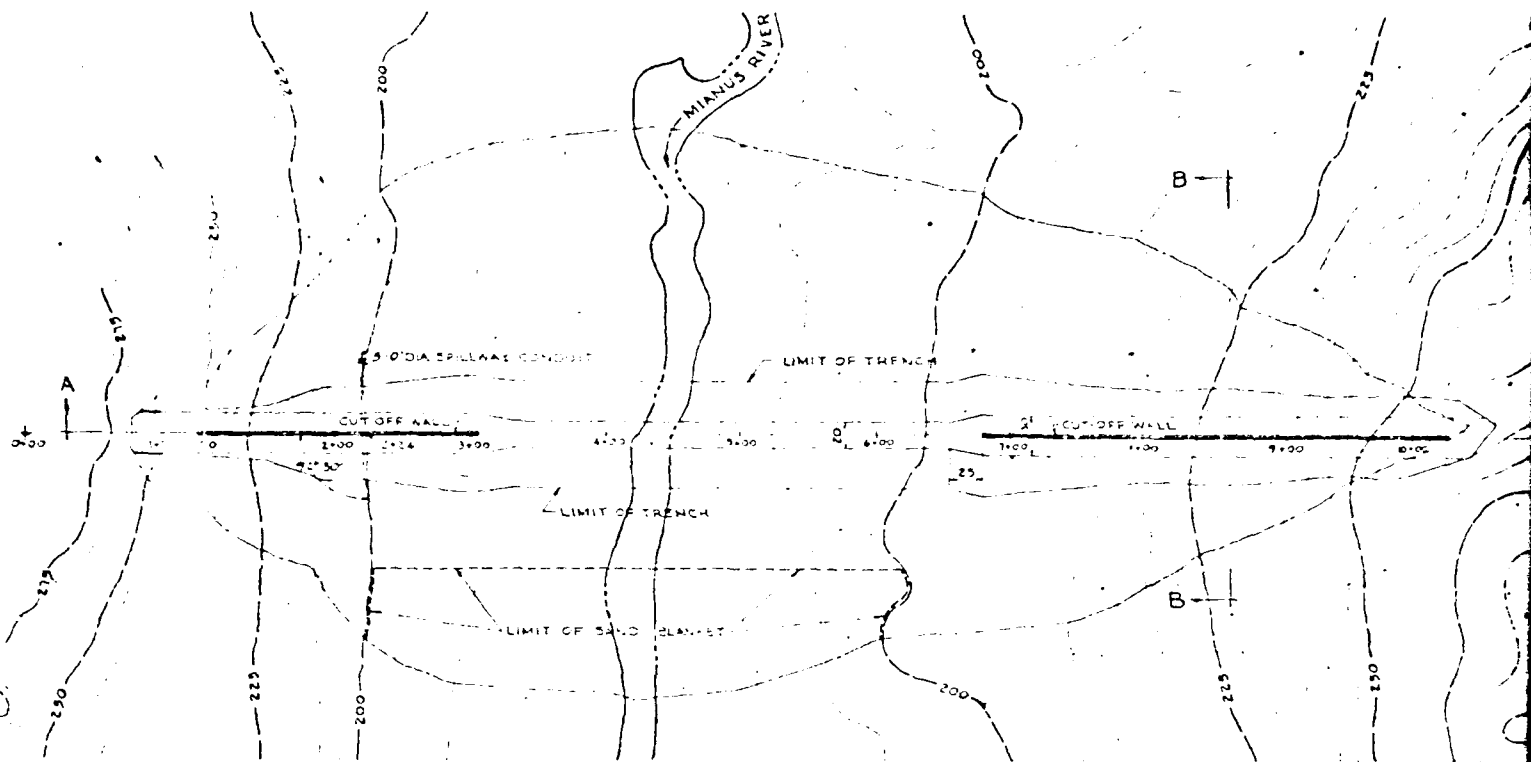
SCALE: 1"=10'-0"



SECTION B-B

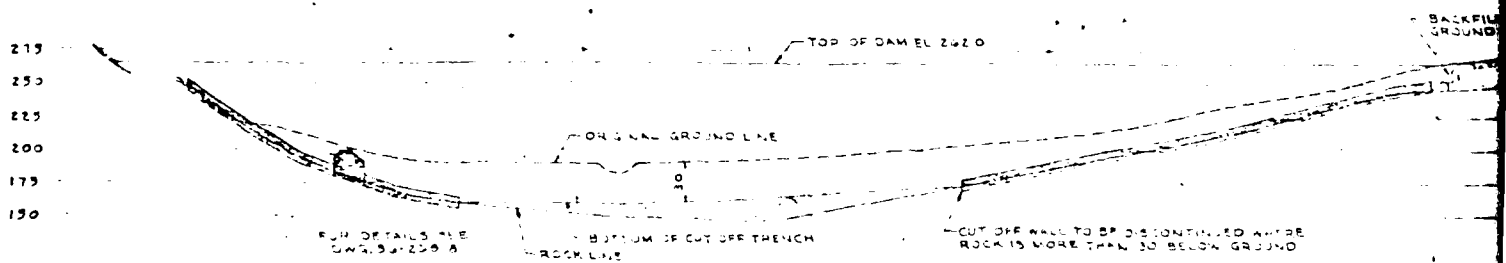
SCALE: 1"=10'-0"

REVISIONS		MIANUS RIVER DAM EMBANKMENT SECTIONS DETAILS	
		GREENWICH WATER COMPANY GREENWICH, N.Y.	
		AMERICAN WATER WORKS SERVICE COMPANY, INC. 121 SOUTH BRAD STREET PHILADELPHIA 5, PA.	
SCALE: AS NOTED		SEE DIMENSIONS ONLY	
DRAWN BY J.M.T.		DATE	
APPROVED		BY	
BY APPROV. ENGINEER		DATE	



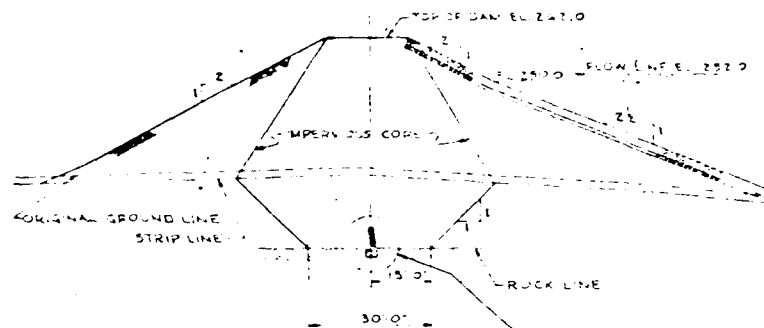
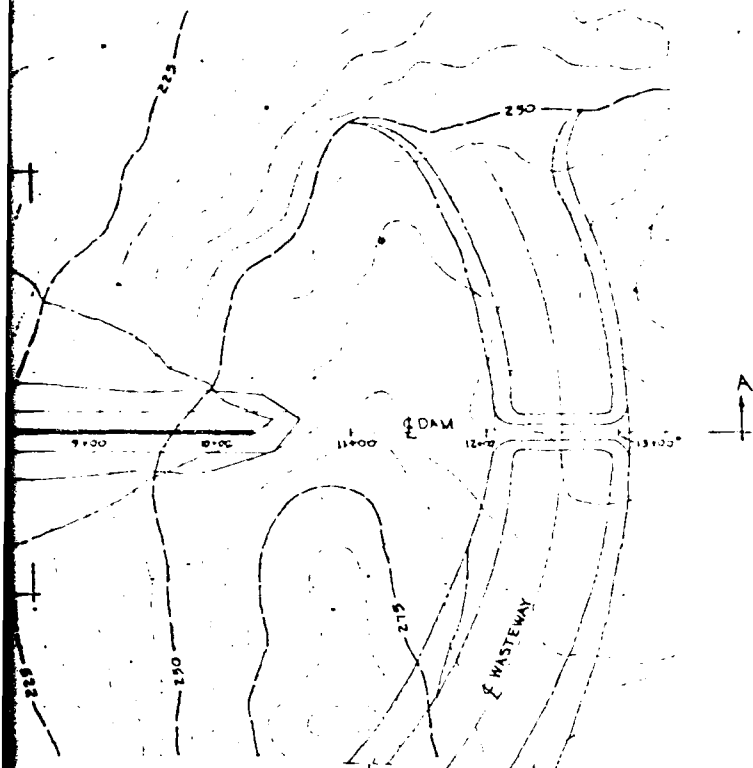
PLAN OF CUT-OFF WALL & TRENCH

SCALE: 1"=60'-0"



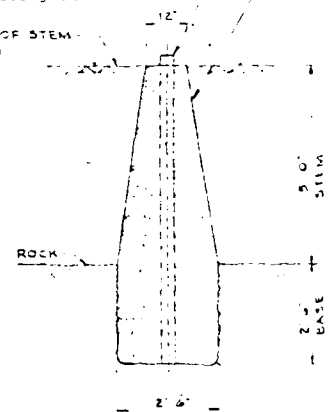
SECTION THRU CUT-OFF WALL AND $\frac{1}{2}$ OF DAM "A-A"

SCALE: 1"=60'-0"

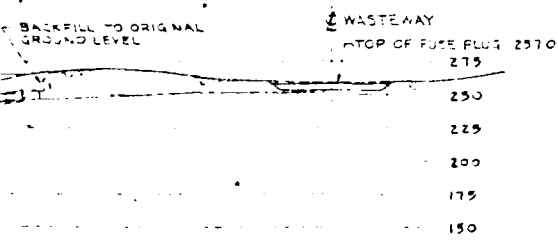


SECTION "B-B"
SCALE: 1" = 20' 0"

2" Ø STD STEEL GROUT PIPE 3'-0" C/C
OR LOCATED AS DIRECTED BY ENGINEER
3" Ø STD STEEL PIPE INSTALLED
FOR WATER TEST HOLES @ 10' CRS
PLACE FILL TO TOP OF STEM
BEFORE GROUTING

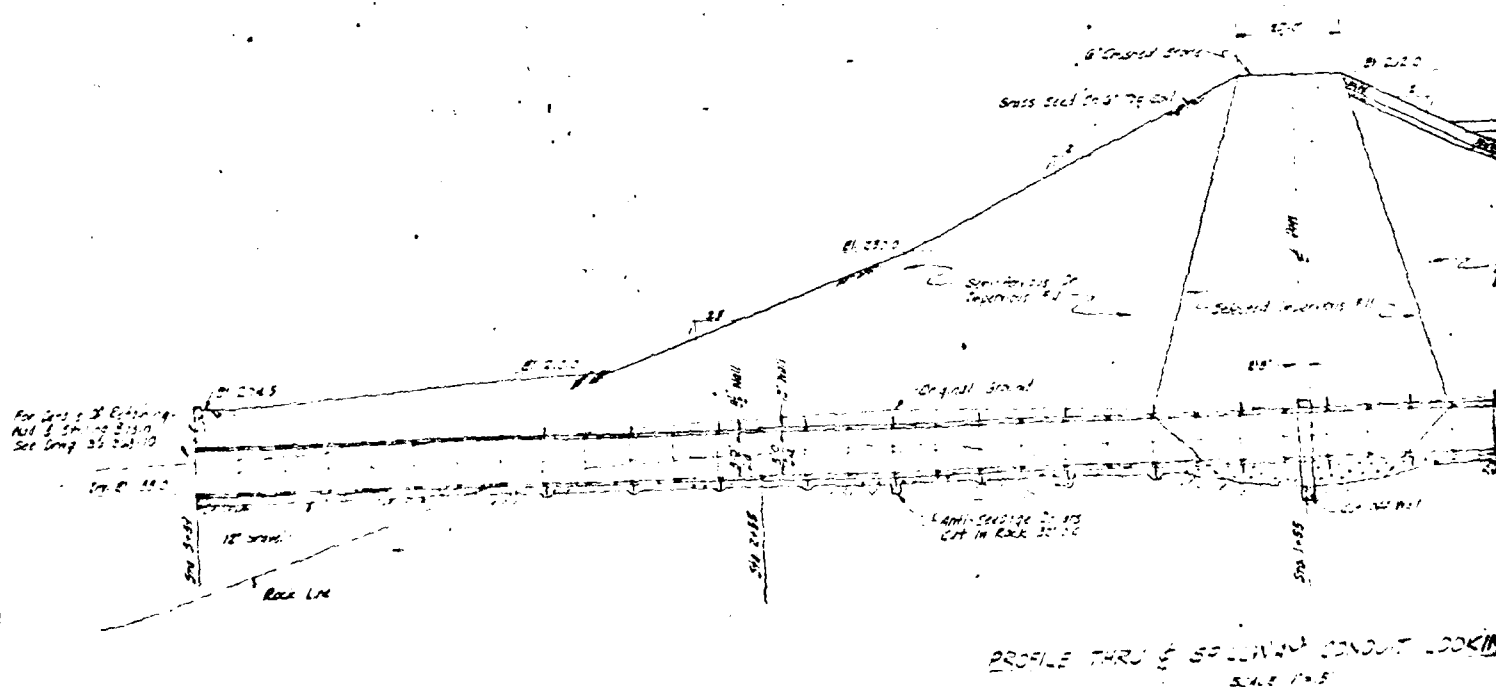
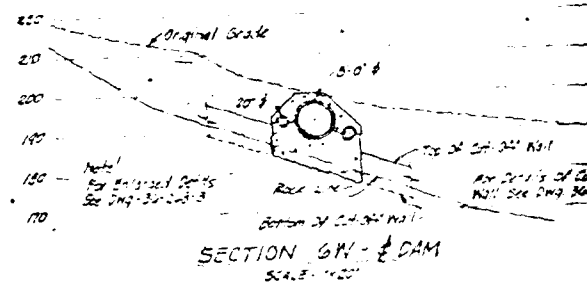
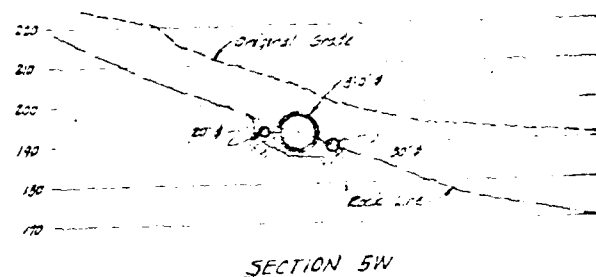
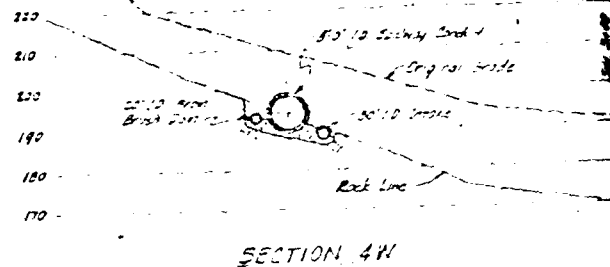
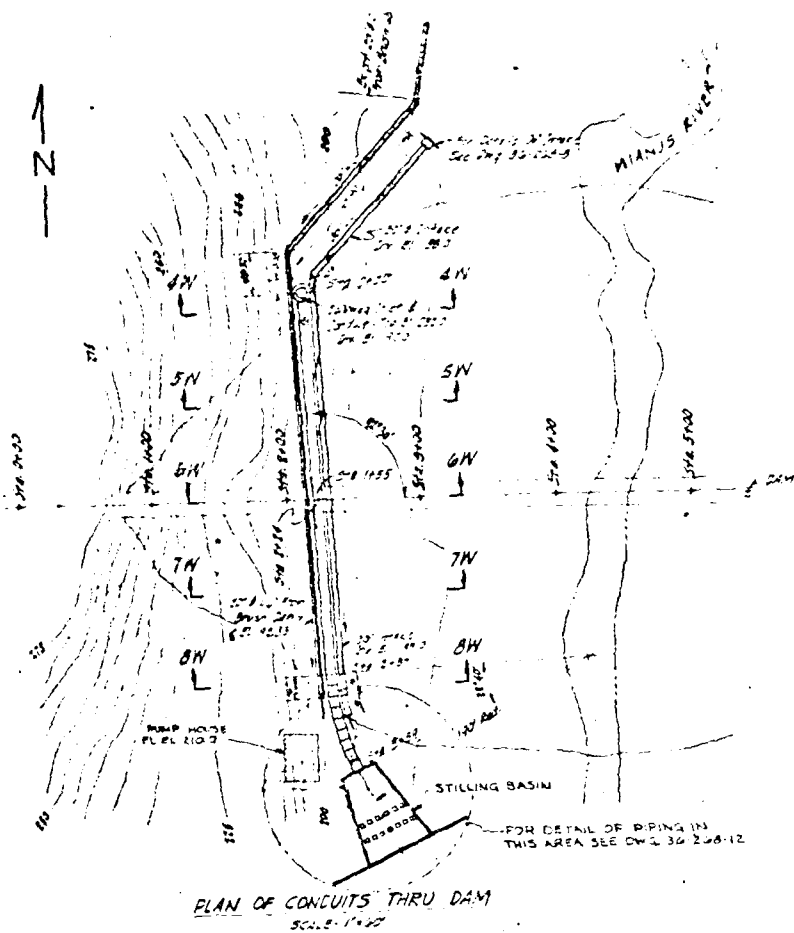


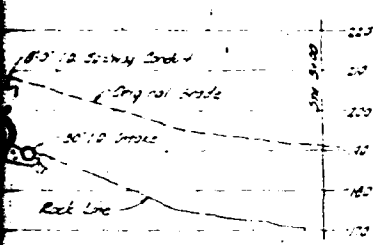
SECTION AT CUT-OFF WALL
SCALE: 1" = 10' 0"



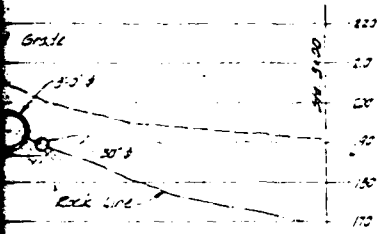
OF DAM "A-A"

REVISIONS	MIANUS RIVER DAM PLAN OF CUT-OFF TRENCH SECTIONS AND DETAILS	
	GREENWICH WATER COMPANY GREENWICH, CONN.	
	AMERICAN WATER WORKS SERVICE COMPANY IN 121 SOUTH BRIDGE STREET PHILADELPHIA 4, PA.	
	SCALE: AS NOTED DRAWN BY: J.R.T. APPROVED: M.L.L.	USE DIMENSIONS ON DATE: 12-2-55 CHECKED BY: PROJECT: P-15 INSP. BY:

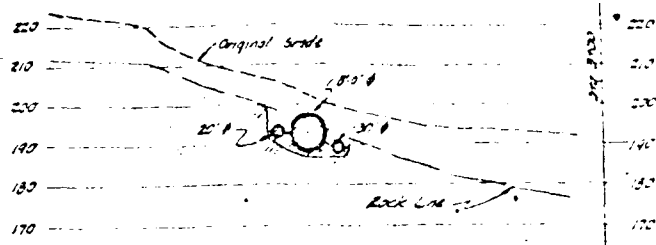




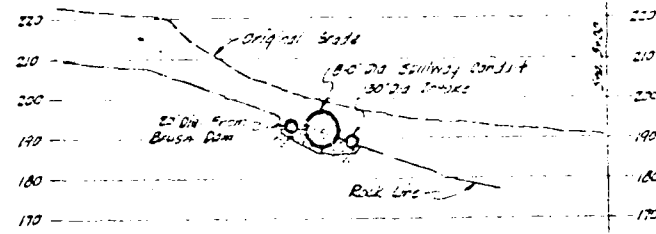
SECTION 4W



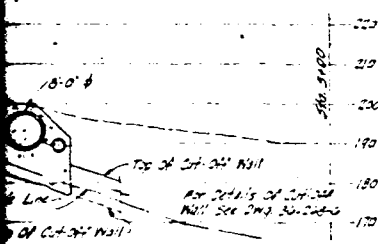
SECTION 5W



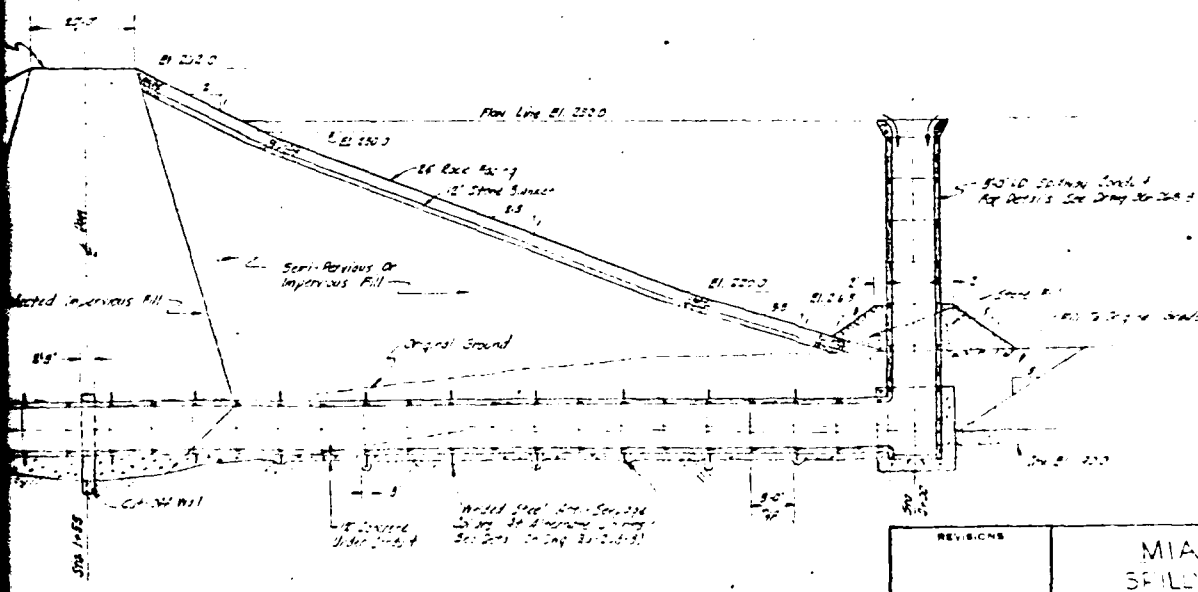
SECTION 7W



SECTION 8W

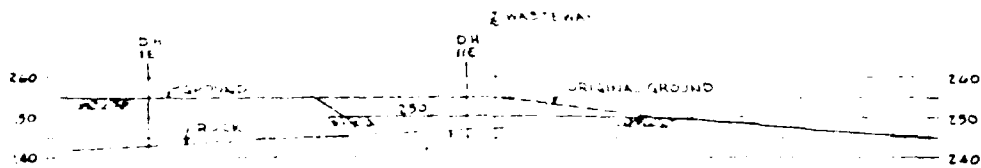


SECTION 9W

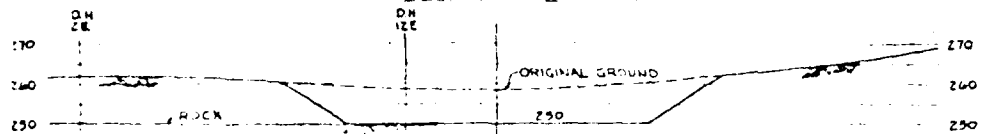


SPILLWAY CONDUIT LOOKING WEST
SCALE 1"=5'

<p>REVISIONS</p>		<p>MIANUS RIVER DAM SPILLWAY INLET & CONDUIT PLAN AND SECTIONS GREENWICH WATER COMPANY GREENWICH, CONN.</p>	
<p>AMERICAN WATER WORKS SERVICE COMPANY, INC. 181 SOUTH BROAD STREET PHILADELPHIA 7, PA.</p>		<p>USE DIMENSIONS ONLY</p>	
<p>SCALE 1"=5'</p>		<p>DATE 12-29-55 CHECKED BY</p>	
<p>DRAWN BY</p>		<p>PROJECT NO. 1000</p>	
<p>APPROVED BY</p>		<p>DESIGNED BY</p>	



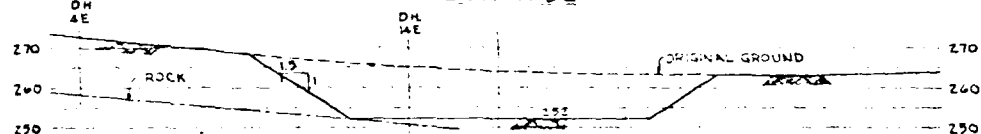
SECTION-1E



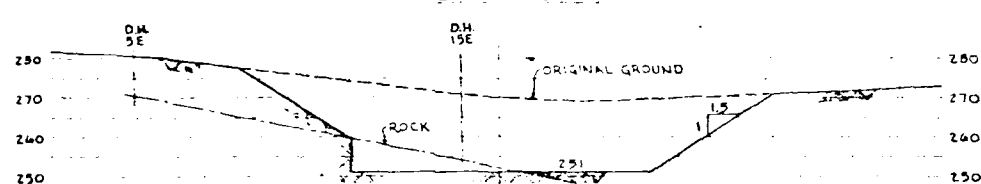
SECTION-2E



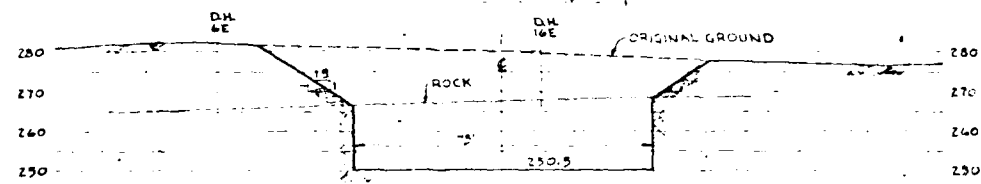
SECTION-3E



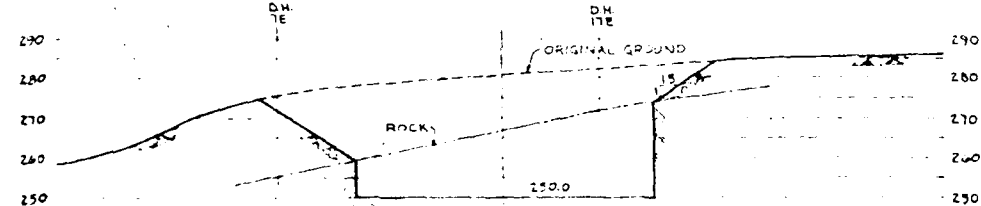
SECTION-4E



SECTION-5E

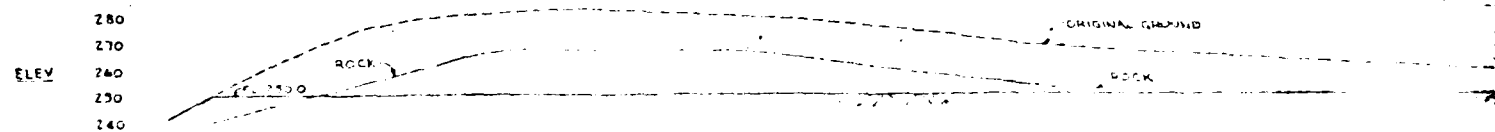


SECTION-6E



SECTION-7E

SCALE: 1" = 20'-0"



PROFILE ALONG E OF WASTE WAY
SCALE: 1" = 30'-0"

METER HOUSE

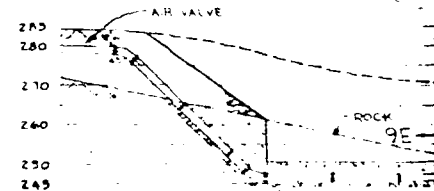
ORIGINAL GROUND AT EX STATION
METER HOUSE STA 3+50

3' TRENCH

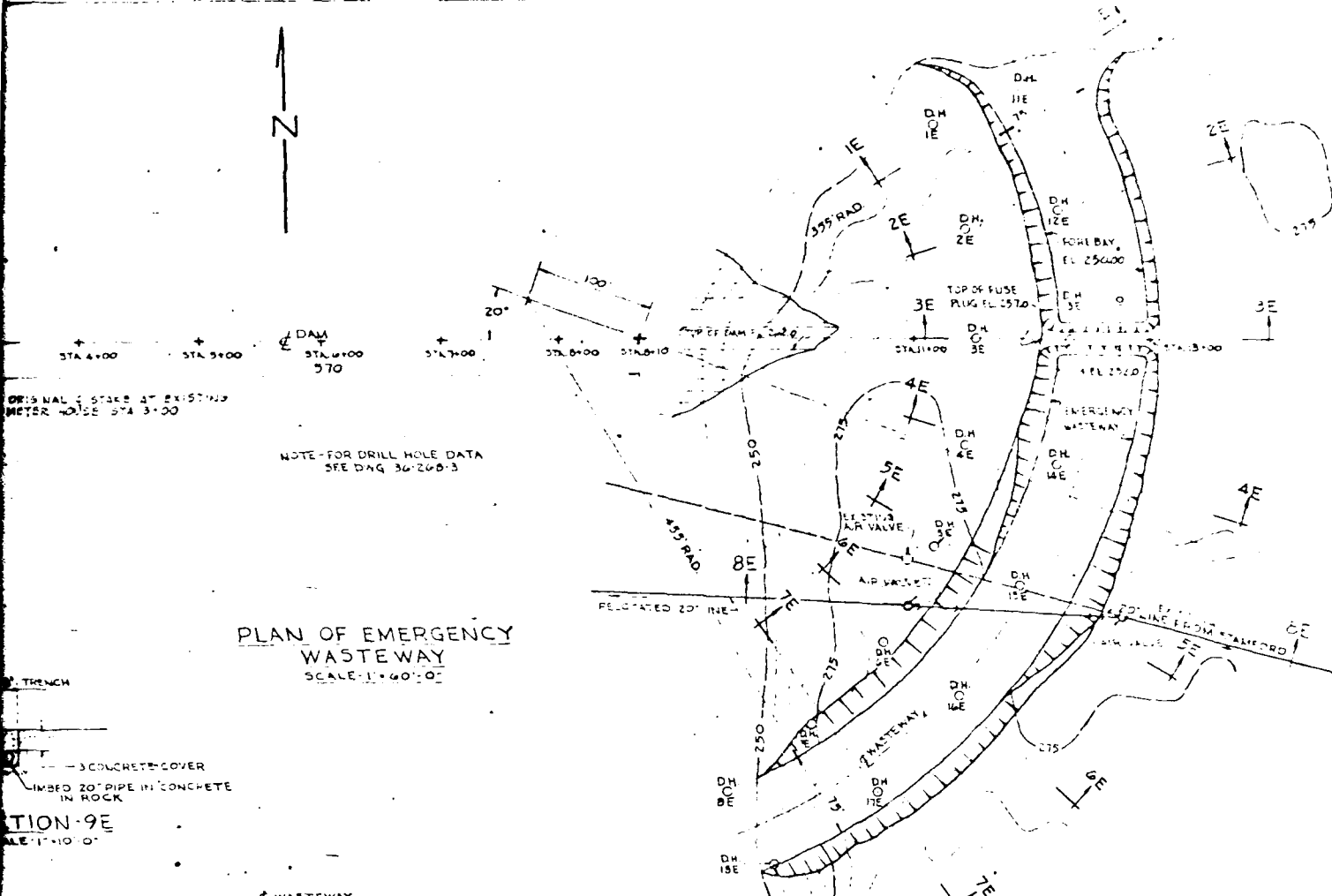
30" RADIUS COVER
LIMITED 20" PIPE IN CONCRETE
IN ROCK

SECTION-9E

SCALE: 1" = 10'-0"

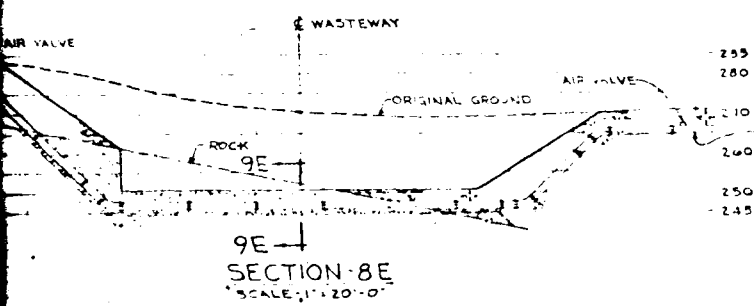


9E
SECT
SCALE

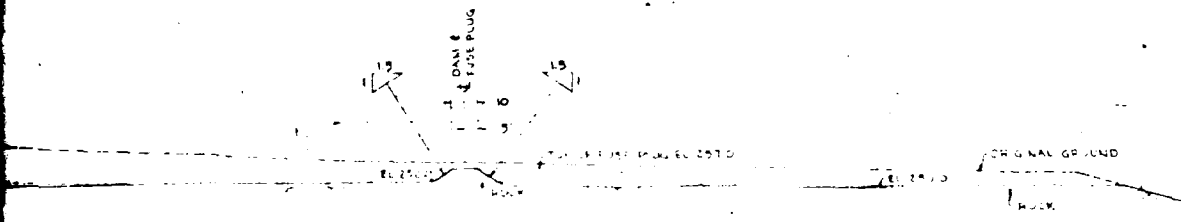


PLAN OF EMERGENCY
WASTEWAY
SCALE: 1"=60'-0"

SECTION 9E
SCALE: 1"=10'-0"



SECTION 8E
SCALE: 1"=20'-0"



SECTION 9E OF WASTEWAY LOOKING WEST
SCALE: 1"=30'-0"

REVISIONS		MIANUS RIVER DAM EMERGENCY WASTEWAY PLAN & SECTIONS	
		GREENWICH WATER COMPANY GREENWICH, CONN.	
		AMERICAN WATER WORKS SERVICE COMPANY INC. 181 SOUTH BROAD STREET PHILADELPHIA 5, PA.	
SCALE AS NOTED		USE DIMENSIONS ONLY	
DRAWN BY J. R. T.		DATE 12-11-53	
APPROVED: M. J. C.		PROJECT P-15	
USE APPROVED DRAWINGS ONLY FOR CONSTRUCTION PURPOSES		36-265-9	

APPENDIX C
PHOTOGRAPHS

MIANUS RESERVOIR DAM
LOCATION OF PHOTOGRAPHS

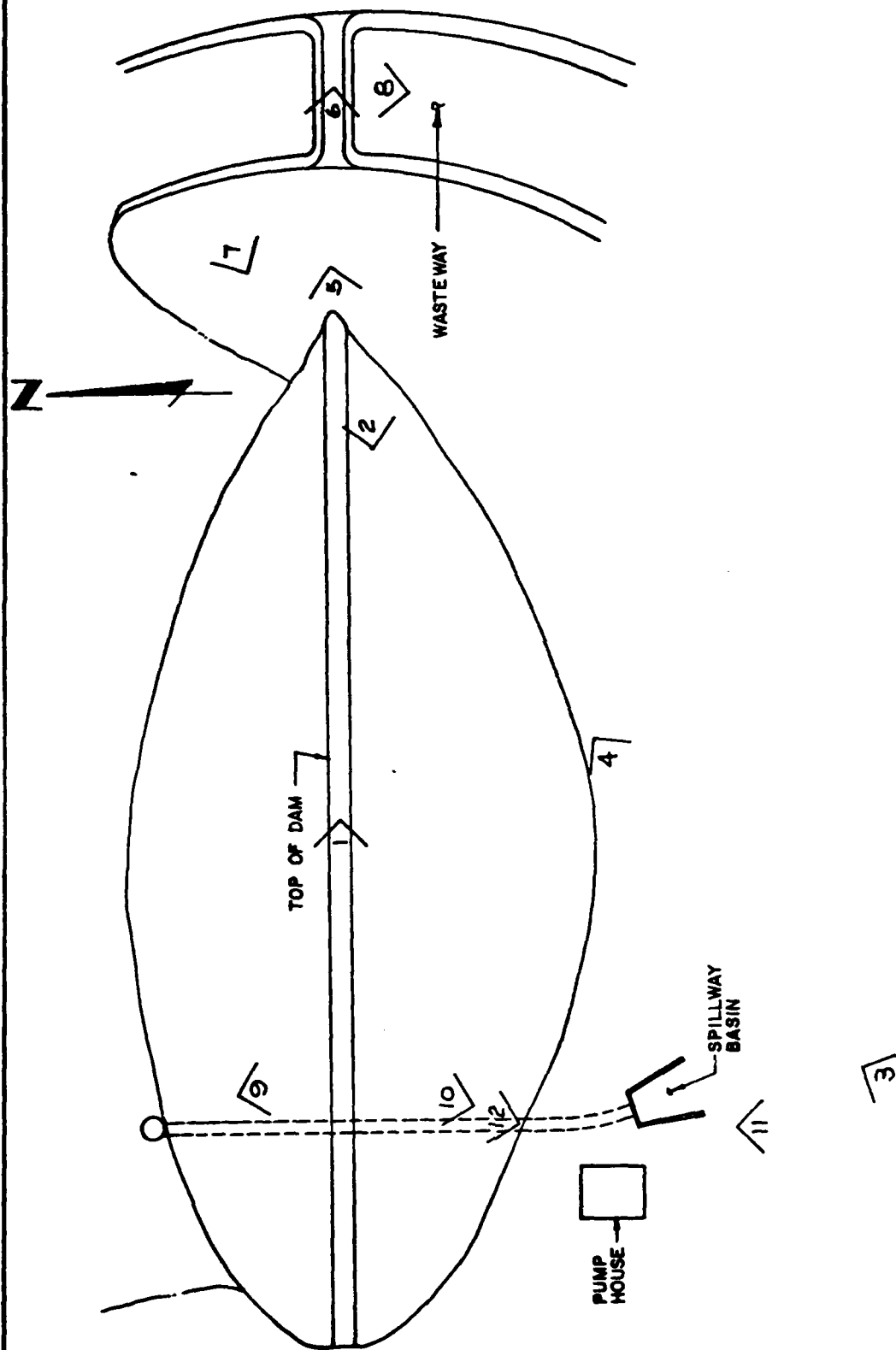




PHOTO #1: Downstream Face of Embankment, looking east.



PHOTO #2: Downstream Face of Embankment, looking west.



PHOTO #3: Front View of Embankment.



PHOTO #4: Toe of Slope, looking east.



PHOTO #5: Looking east toward fuse plug.

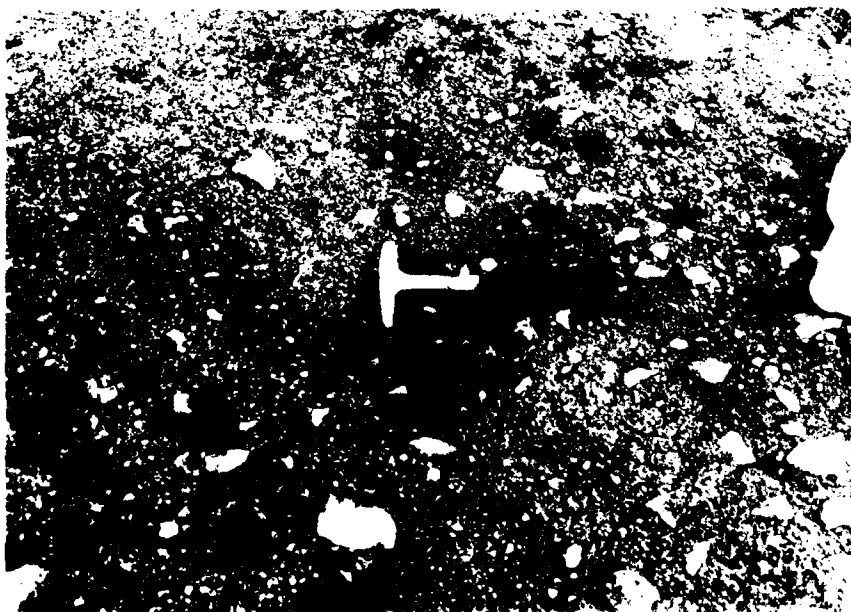


PHOTO #6: Small test pit in center of fuse plug.

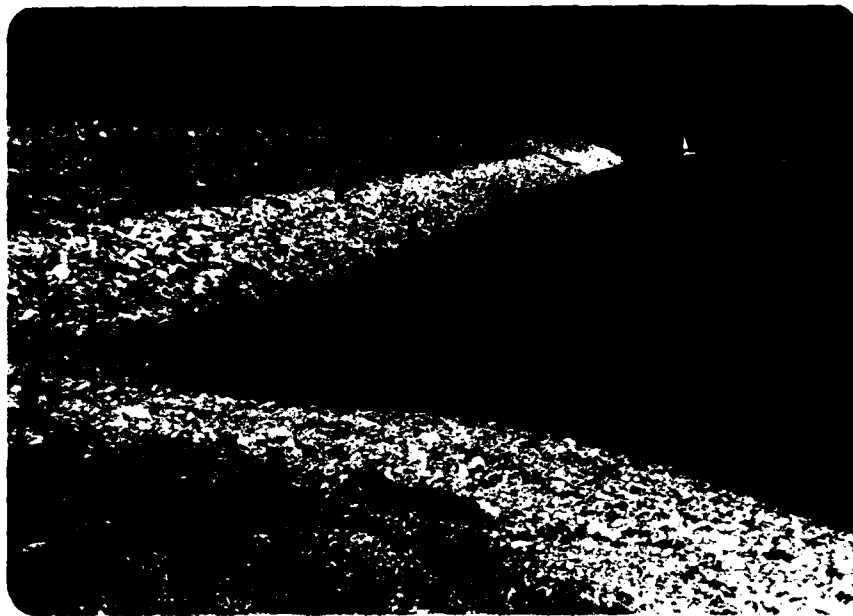


PHOTO #7: Upstream Face of Dam, looking west.



PHOTO #8: Emergency spillway (fuse plug) discharge channel,
looking downstream.

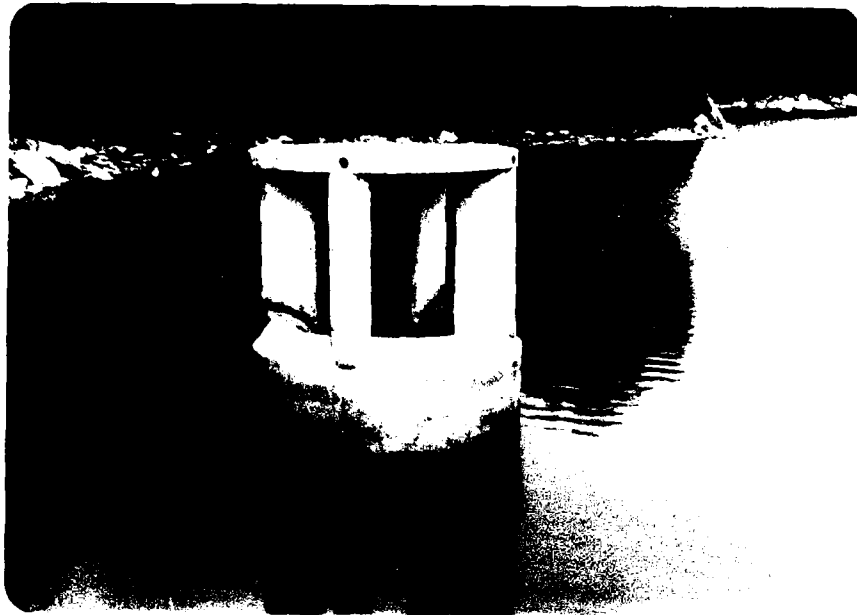


PHOTO #9: Service spillway - Top of the drop inlet shaft.



PHOTO #10: Service spillway discharge channel, looking downstream.

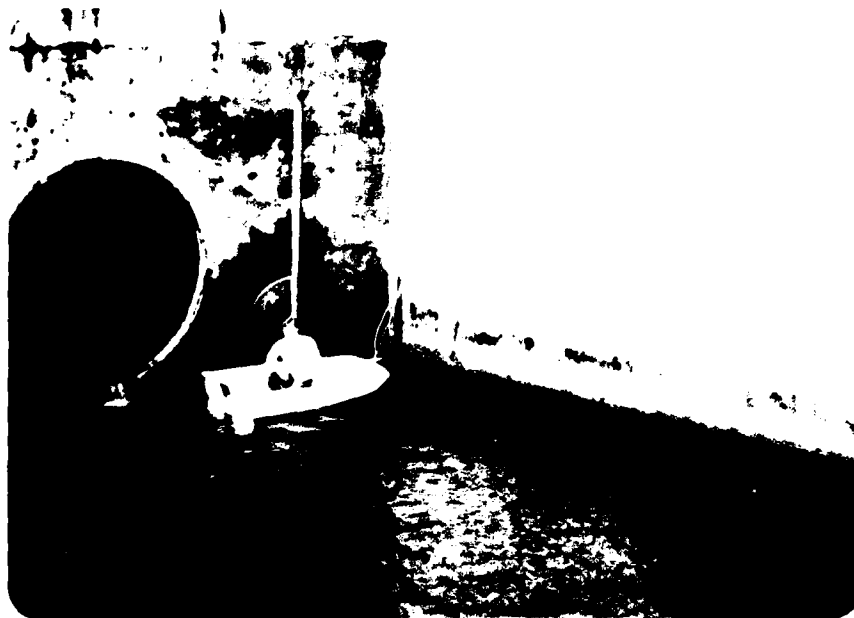


PHOTO #11: Outlet of the service spillway discharge conduit,
and adjacent blow off.

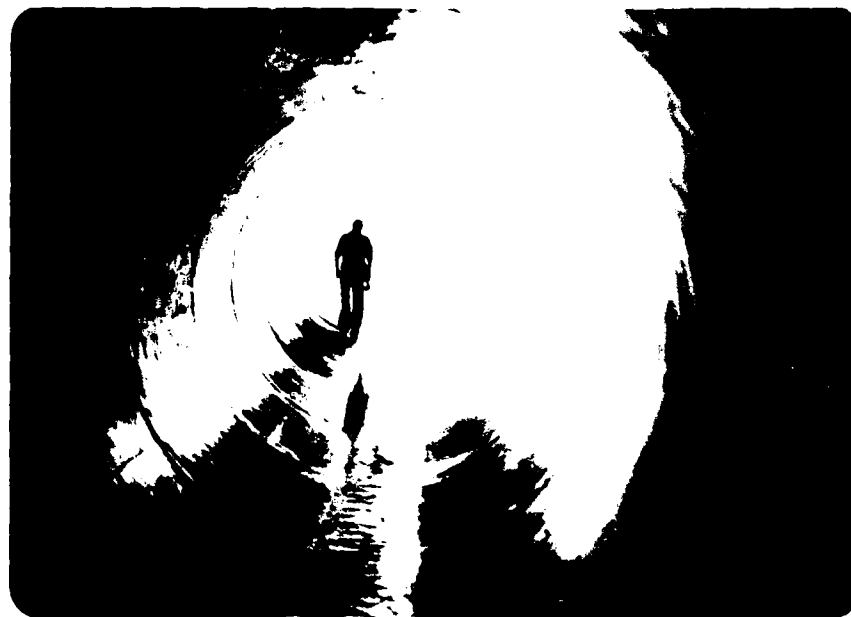


PHOTO #12: Interior view of the service spillway discharge
conduit.

AD-A142 692

NATIONAL DAM INSPECTION PROGRAM MIANUS RESERVOIR DAM
(CT 00050) CONNECTIC..(U) CORPS OF ENGINEERS WALTHAM MA
NEW ENGLAND DIV SEP 78

2/2

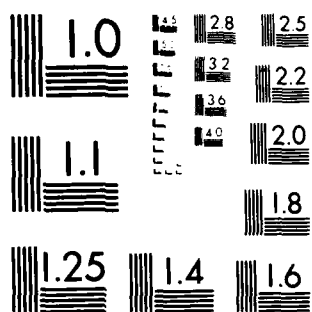
UNCLASSIFIED

F/G 13/13

NL

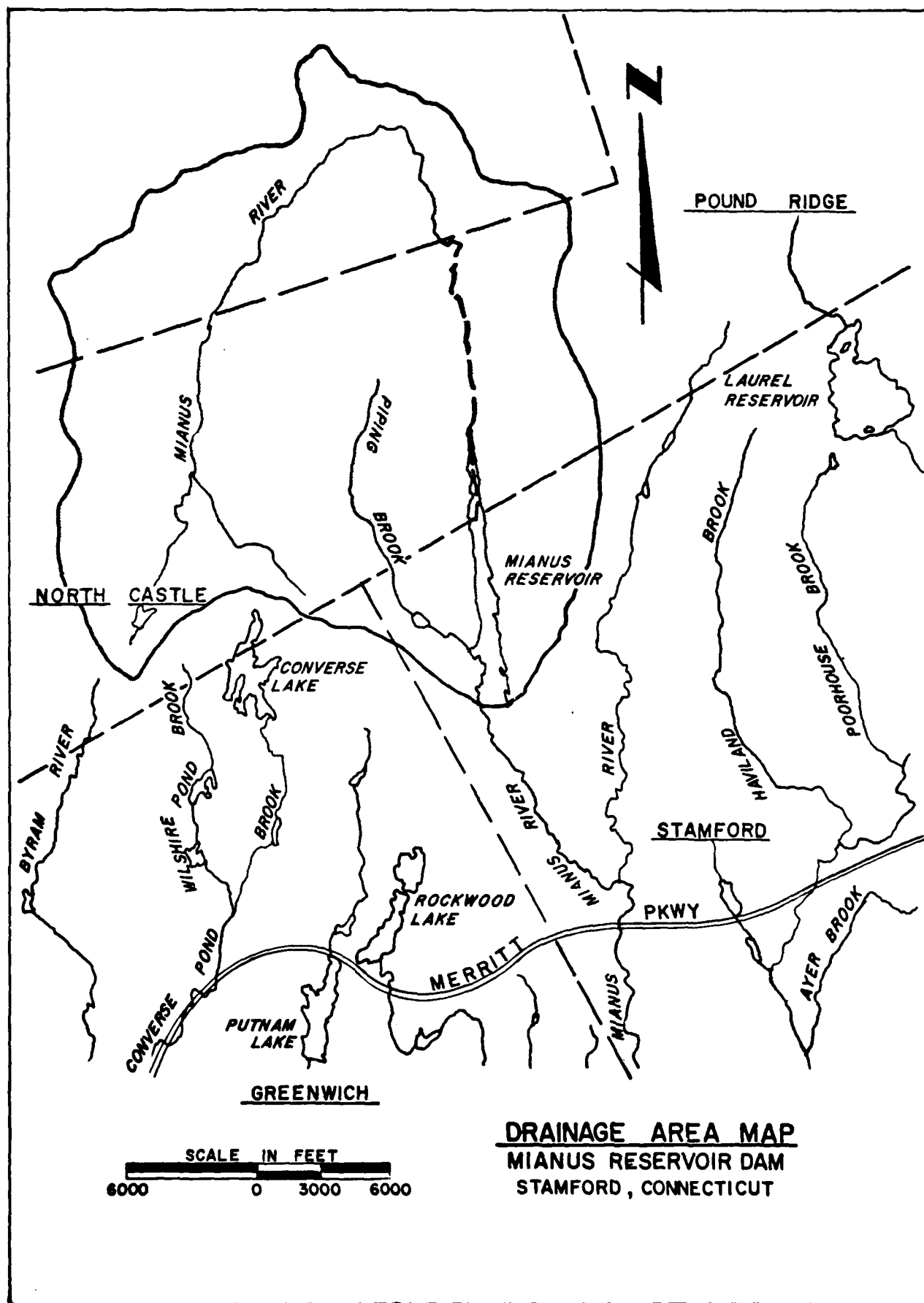


END
DATE
FILMED
8-84
DTIC



MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A

APPENDIX D
HYDROLOGIC COMPUTATIONS



78-36-10
US DAM



FLAHERTY-GIAVARA ASSOCIATES
ENVIRONMENTAL DESIGN CONSULTANTS
ONE COLUMBUS PLAZA, NEW HAVEN, CONN 06510/203/789-1280

SHEET NO. 1 OF 1
BY JGM DATE 8/10/75
CHK'D BY EJR DATE 8/29/75

P.M.F. PEAK FLOW ESTIMATE

WATERSHED AREA IS 18.3 SQ MILES

METHOD #1

REFER TO "PRELIMINARY GUIDANCE FOR
ESTIMATING PMF DISCHARGES" BY NEW
ENGLAND DIVISION, CORPS OF ENGINEERS

$$\begin{aligned}\text{UNIT FLOW} &= 1550 \text{ CFS} / \text{mi}^2 \quad (\text{ROLLING Curve}) \\ \text{PMF} &\approx 18.3 \text{ mi}^2 \times (1550 \text{ CFS} / \text{mi}^2) = 28,365 \text{ CFS} \\ \text{SAY} &\approx 28,000 \text{ CFS}\end{aligned}$$

METHOD #2

REFER TO \approx CONN WATER RESOURCE
BULLETIN #17, PART 4, BY U.S.G.S.

$$\begin{aligned}\text{MEAN ANNUAL FLOOD} &= 700 \text{ CFS} \quad (\text{FIG. 13}) \\ Q_{100} &= 5 \times \text{MAF} = 5 \times 700 \text{ CFS} = 3500 \text{ CFS} \\ \text{PMF} &\approx 5 \times Q_{100} \quad (\text{APPROXIMATE}) \\ \text{PMF} &\approx 5 \times 3500 \text{ CFS} \approx 17,500\end{aligned}$$

METHOD #3

REFER TO FAIRFIELD, CT., F.I.A. FLOOD
INSURANCE STUDY \approx FREQUENCY, DISCHARGE,
DRAINAGE AREA CURVES"

$$\begin{aligned}Q_{100} &= 4500 \text{ CFS} \quad (\text{FIG. 2}) \\ \text{PMF} &\approx 5 \times 4500 \text{ CFS} = 22,500 \text{ CFS}\end{aligned}$$

FOR SPILLWAY TEST FLOOD, USE 28,000

78-36-10
MIANUS DAM



FLAHERTY-GIAVARA ASSOCIATES
ENVIRONMENTAL DESIGN CONSULTANTS
ONE COLUMBUS PLAZA, NEW HAVEN, CONN. 06510/203/789-1280

SHEET NO. 2 OF 8
BY JGM DATE 8/10/78
CHK'D BY EJR DATE 8/29/78

FORMATION OF INFLOW HYDROGRAPH

- 1) TEST FLOOD = 28,000 CFS
- 2) FORM A TRIANGULAR HYDROGRAPH, WITH
24 HOUR DURATION, PEAK AT 8 HOURS

<u>TIME</u> <u>HOURS</u>	<u>UNIT FLOW</u> <u>RATE</u>	<u>FLOW RATE</u> <u>CFS</u>
0	0.00	0
2	0.25	7,000
4	0.50	14,000
6	0.75	21,000
8	1.00	28,000
10	0.875	24,500
12	0.75	21,000
16	0.50	14,000
20	0.25	7,000
24	0.00	0

18-36-10
MIANUS



FLAHERTY-GIAVARA ASSOCIATES
ENVIRONMENTAL DESIGN CONSULTANTS
ONE COLUMBUS PLAZA, NEW HAVEN, CONN 06510/203/789-1280

SHEET NO. 5 OF 5
BY JGM DATE 8/10/78
CHK'D. BY EJR DATE 8/24/78

SERVICE SPILLWAY

"MORNING GLORY" TYPE WITH 12 FT
DIA WEIR, 8 FT DIA DROP SHAFT
AND CONDUIT, $R_s = 6$ FT

$L = \text{PERIMETER} - \text{PIER WIDTH}$

$$L = 12'(\pi) - 4(1.5') = 31.7 \text{ FT}$$

H_o	H_o/R_s	C_o	Q (CFS)	STAGE
0	0		0	252
1	0.16	3.90	124	253
2	0.33	3.70	332	254
3	0.50	3.35	552	255
4	0.67	2.82	715	256
5	0.83	2.37	840	257
6	1.00	2.02	941	258
7	1.16	1.77	1039	259
8	1.33	1.52	1090	260
9	1.50	1.36	1164	261
10	1.67	1.22	1223	262

THE ABOVE ESTIMATED SERVICE SPILLWAY
DISCHARGE RATES ARE BASED ON
DATA FROM CHAP. IX, SECTION F, 212
OF "DESIGN OF SMALL DAMS", U.S.
DEPT. OF THE INTERIOR, 1973.

WHERE:

$$Q = C_o L H^{3/2}$$

78-36-10

MIANUS



FLAHERTY-GIAVARA ASSOCIATES
 ENVIRONMENTAL DESIGN CONSULTANTS
 ONE COLUMBUS PLAZA, NEW HAVEN, CONN. 06510/203/789-1280

SHEET NO. 4 OF 8
 BY JGM DATE 8/10/78
 CHK'D BY EJR DATE 8/29/78

AUXILLARY SPILLWAY

CREST WILL ACT AS A BROAD CRESTED
 WEIR, COEFFICIENT $C = 3.0$
 EFFECTIVE LENGTH IS 90 FEET \pm .
 ANALYZE WITH AND WITHOUT
 FUSE PLUG IN PLACE.

STAGE ELEVATION	<u>H_o</u>	W/FUSE Plug <u>Q CFS</u>	W/O Fuse PLUG <u>H_o</u>	<u>Q CFS</u>
252			0	0
253			1	270
254			2	764
255			3	1403
256			4	2160
257	0	0	5	3019
258	1	270	6	3968
259	2	764	7	5000
260	3	1403	8	6109
261	4	2160	9	7290
262	5	3019	10	8538

MIANUS DAM

78-36-10

FLOOD ROUTING

JGM

9/19/78

INPUT DATA:

SEGMENT 1

SEGMENT 2

SEGMENT 3

IR-252 IV-

UNSUBMERGED WEIR

DISCHARGE COEFFICIENT = 2

DISCHARGE COEFFICIENT = 3

DISCHARGE COEFFICIENT = 3

0.0 E=252 A=230.00

2

3

3

E=260 A=240.00

LENGTH OF WEIR = 31.7

LENGTH OF WEIR = 90

LENGTH OF WEIR = 860

E=270 A=240.00

ELEVATION OF WEIR = 252

ELEVATION OF WEIR = 252

ELEVATION OF WEIR = 262

HOOR	INFLOW	MASS INFLOW	WATER EL.	TAIL WATER	OUTFLOW	MASS OUTFLOW	STORAGE(R)	STORAGE(A)
0.00	0CFS	0.00AC-F	252.00FT	0.00FT	0CFS	0.00AC-F	0.00AC-F	0.00AC-F
2.00	7.000CFS	578.51AC-F	254.13FT	0.00FT	1.036CFS	85.67AC-F	492.83AC-F	492.83AC-F
4.00	14.000CFS	2,314.04AC-F	258.97FT	0.00FT	6,143CFS	679.07AC-F	1,634.97AC-F	1,634.97AC-F
6.00	21.000CFS	5,206.51AC-F	263.27FT	0.00FT	16,366CFS	2,539.41AC-F	2,667.19AC-F	2,667.19AC-F
8.00	28.000CFS	9,256.19AC-F	264.87FT	0.00FT	27,995CFS	6,205.73AC-F	3,050.46AC-F	3,050.46AC-F
10.00	24.500CFS	13,595.04AC-F	264.56FT	0.00FT	25,420CFS	6,202.27AC-F	2,974.76AC-F	2,974.76AC-F
12.00	21.000CFS	17,355.37AC-F	264.05FT	0.00FT	21,551CFS	5,502.27AC-F	2,853.09AC-F	2,853.09AC-F
16.00	14.000CFS	23,140.49AC-F	263.03FT	0.00FT	14,930CFS	5,324.42AC-F	2,608.07AC-F	2,608.07AC-F
20.00	7.000CFS	26,611.57AC-F	261.00FT	0.00FT	9,012CFS	4,489.90AC-F	2,121.66AC-F	2,121.66AC-F
24.00	0CFS	27,768.59AC-F	257.01FT	0.00FT	3,746CFS	26,598.86AC-F	1,169.73AC-F	1,169.73AC-F
30.00	0CFS	27,768.59AC-F	252.79FT	0.00FT	234CFS	27,586.10AC-F	182.49AC-F	182.49AC-F
36.00	0CFS	27,768.59AC-F	252.43FT	0.00FT	96CFS	27,668.14AC-F	100.44AC-F	100.44AC-F

APPENDIX E

INFORMATION - NATIONAL INVENTORY OF DAMS



INVENTORY OF DAMS IN THE UNITED STATES

IDENTITY NUMBER	DIVISION	STATE	COUNTY	DIST.	CONSTR. DIST.	NAME	LATITUDE (NORTH)	LONGITUDE (WEST)	REPORT DATE DAY MO YR
CT	50	NEO	CT	001	04	MIANUS RESERVOIR DAM	4107.8	7356.7	08 SEP 78

POPULAR NAME	NAME OF IMPOUNDMENT
SAMUEL J BARCHM RESERVOIR DAM	MIANUS RESERVOIR

REGION/BASIN	RIVER OR STREAM	NEAREST DOWNSTREAM CITY - TOWN - VILLAGE	DIST FROM DAM (MI)	POPULATION
01 07	MIANUS RIVER	RIVERBANK	2	1000

TYPE OF DAM	YEAR COMPLETED	PURPOSES	STRUCTURAL HEIGHT (FT)	HYDRAULIC HEIGHT (FT)	IMPOUNDING CAPACITIES (ACRES-FT)	DIST UN N	FEU	PRV/FED	SCS	A	VEH/DATE
HEPU	1955	S	75	75	7515	NEU	N	N	N	N	28 AUG 78

REMARKS											
D.S. SPILLWAY											
HAS LENGTH	TYPE	WIDTH	HEIGHT	VOLUME OF DAM (CUY)	MAXIMUM DISCHARGE (FT)	POWER CAPACITY (MW)	INSTALLED	PROPOSED	NO LENGTH	WIDTH	HEIGHT
1	950	U	90	5000							
OWNER											
ENGINEERING BY											
CONSTRUCTION BY											
CON/AMERICAN WATERWORKS											
AMERICAN WATERWORKS SERV											
HERMITT CHAPMAN SCOTT											
REGULATORY AGENCY											
CONSTRUCTION											
OPERATION											
MAINTENANCE											
NONE											
INSPECTION BY											
INSPECTION DATE											
DAY MO YR											
AUTHORITY FOR INSPECTION											
FLAMERTY GIAVARA ASSOC, PC											
05 AUG 78											
PL 92-367											
REMARKS											